Multnomah County

Land Use Planning Division
1600 SE 190th Ave, Ste 116
Portland OR 97233
Ph: 503-988-3043 Fax: 503-988-3389
multco.us/landuse

HILLSIDE DEVELOPMENT PERMIT (HDP) APPLICATION:
GEOTECHNICAL RECONNAISSANCE AND STABILITY PRELIMINARY STUDY

Note: Response to each question below must be completed or verified by a Certified Engineering Geologist or Geotechnical Engineer, including a State of Oregon Registration Stamp and Number in the space provided on page four. The HDP form 1 addresses Multnomah County Code Section .5515(A)(3), Hillside Development Permits.

Site Address: 40301 East Larch Mountain Road, Corbett, OR 97019

Legal Description: 1N-5E-30CC, Tax Lot 1500 & 1600

Property Owner's Name: HSF, LLC

Firm Preparing Report: Geotechnical Resources, Inc.

Address: 9750 SW Nimbus Avenue

City: Beaverton **State:** OR **Zip:** 97008

Preparer's Name: Wes Spang, George Freitag

Phone Number: 503-641-3478

GENERAL PROPERTY INFORMATION

1. a. Maximum Slope on Property: 60 degrees Area in which it is located: west property margin
   Average Slope of Property: 10 degrees on remainder of property

   b. Are there any wetlands or streambeds on the property? (Please Circle) Yes (No)
   If yes, please show on topographical survey or sketch.

   c. Volume of soil or earth material disturbed, stored, disposed of or used as fill: 299 CY

   d. Total area of proposed ground disturbance:
   28,850 (square feet) 0.66 (acres)

EXHIBIT
I.10

Were building plans considered when completing this form? (Please Circle) (Yes) No

If yes, please note the author and date the plans were prepared.

Emerick Architects 4/24/19

2. What is the general topography of the property? Please attach a topographic survey or sketch with pertinent notes.

The site consists of two tax lots separated by a public road at elevation of 960.0'. The parking lot of TL 1500 will be adjusted from an existing 2.0% slope falling to northeasterly elevation 956.0' to a proposed 3.2% slope with 3:1 catch slopes at north and east boundaries. TL 1600 has a northwesterly fall sloping 2.0% across existing building pad then a variable 5% to 25% slope to top of bluff at 946.0'. The most northwesterly 20.0' to 30.0' of property falls further down from the top of bluff towards the Historic Columbia River Highway and was not surveyed. See attached survey.

3. Are there any visible signs of instability or other potentially adverse site features (Landslides, slumps, mud flow, creep, ravines, fills, cuts, seeps, springs, ponds, etc.) within the surrounding area for a minimum distance of 100 feet beyond the subject property boundaries? Describe and indicate on attached topographic survey or sketch.

The slope along the western property margin, adjacent to the east side of the Historic Columbia River Highway, is mapped by DOGAMI as the head scarp of the large landslide that is present west of the highway (labeled Washougal_103 by DOGAMI). The head scarp is present along the east side of the highway from E. Larch Mountain Road north to the Vista House. Additional information to be included in the geotechnical report.

4. Is any earthwork proposed in connection with site development?

(Please Circle) (Yes) No

If yes, please indicate depth and extent of cuts/fills; describe fill types.

A variable cut from 3' to 10' will occur within the proposed building outline in order to excavate the existing basement to the proposed 3916 sq ft basement, FF to 948.0'. Additional cut and fill on site will be to place new gravel for the paths, loading zone and parking lot.

5. In your opinion, will the proposed earthwork cause potential stability problems for the subject and/or adjacent properties?

(Please Circle) Yes (No)

IF YES, EXPRESS PROBABILITY:

(Please Circle) Very Probable Possibly Possible, but remote

If Very Probable or Possibly, please explain.

6. In your opinion, will the proposed development (structures, foundations, parking area, streets, etc.) create potential stability problems for the subject and/or adjacent properties?

(Please Circle) Yes (No)

IF YES, EXPRESS PROBABILITY:

(Please Circle) Very Probable Possibly Possible, but remote

If Very Probable or Possibly, please explain.

7. In your opinion would the subsurface disposal of sewage effluent on the site (i.e., drain fields) have an adverse affect on stability of the site or adjacent area?

(Please Circle) (Yes) No

IF YES, EXPRESS PROBABILITY:

(Please Circle) Very Probable Possibly (Possible, but remote)

If Very Probable or Possibly, please explain.

This assumes there is minimal increase in effluent flow to existing drain field.

8. If answer is Very Probable or Possibly to questions 4 or 5, is it your opinion, on the basis of a visual evaluation, that adequate stability might be achieved by preferred siting of the development, alternative foundation support, earthwork, drainage, etc.?

(Please Circle) (Yes) No

If yes, please explain.

Proposed building is ~ 125 ft from the top of slope present along the western property line. Minimal fill being placed on site.

9. Do you recommend additional geotechnical studies (i.e., mapping, testing pits or borings, stability analysis, etc.) prior to site development?

(Please Circle) (Yes) No

If yes, please explain.

A Geotechnical engineering investigation and report was completed by GRI on 12/5/18.

By signing and affixing the required stamp below, the Certifying Engineering Geologist or Geotechnical Engineer certifies that the site is suitable for the proposed development.

Signature Wesley Spang

Date 5-23-19

Affix Seal Here

REGISTERED PROFESSIONAL ENGINEER
18281
OREGON
JAN 16, 1998
WESLEY SPANG

Renew 06-2020

GRI
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May 24, 2019

6060 GEOTECHNICAL CONSULTATION

HSF, LLC
c/o Emerick Architects P.C.
321 SW 4th Avenue, Suite 200
Portland, OR 97204

Attention: Keith Daily

SUBJECT: Geotechnical Consultation
View Point Inn
40301 E Larch Mountain Road
Corbett, Oregon

This letter provides an additional geotechnical consultation performed for the View Point Inn in Corbett, Oregon. This letter presents the results of our review of the Multnomah County Land Use Planning Division Hillside Development Permit (HDP) Work Sheet prepared by Humber Design Group, Inc.

BACKGROUND INFORMATION

GRI completed a geotechnical investigation for the project, the results of which were provided to HSF, LLC in our December 5, 2018, report titled, "Geotechnical Engineering Investigation, View Point Inn, 40301 E Larch Mountain Road, Corbett, Oregon."

CONSULTATION

GRI reviewed the HDP prepared by Humber Design Group, Inc., dated May 14, 2019 (attached). The HDP provides information regarding site topography and slopes, proposed grading, stormwater drainage and erosion control, and other civil engineering design criteria.

The geotechnical engineering information provided in the HDP is consistent with the discussion, conclusions, and recommendations presented in our December 5, 2018, geotechnical report for the site.

This consultation letter is subject to the same limitations discussed in our December 5, 2018, geotechnical report.

Please contact the undersigned if you have any questions.

Submitted for GRI,

REGISTERED PROFESSIONAL ENGINEER
18281
OREGON
JAN 16, 1998
WESLEY SPANG

Renews 06/2020

A. Wesley Spang, PhD, PE, GE
Principal

EXHIBIT
I.11

Land Use Planning Division
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Hillside Development Permit (HDP) Work Sheet

Associated Active Cases:

Instructions for Applicants:

This questionnaire has been put together to assist you in preparing an application for development within the Hillside Development Overlay. While not required, we encourage you to consult with an Oregon licensed Certified Engineering Geologist or Geotechnical Engineer when completing this form. Information in this worksheet is intended to supplement the Geotechnical Report or Geotechnical Reconnaissance Survey [HDP Form 1]. The responses and supporting documents you provide will be the basis for determining whether or not your application satisfies the Hillside Development criteria.

GENERAL INFORMATION

Project Description: Renovation of fire damaged View Point Inn to become a spa/retreat facility.

Site Address or Legal Description: 40301 E Larch Mtn Rd, Corbett, OR Average Slope of Property (%): 10%

Maximum Slope on Property (%): 33% Proposed Area in which it is located: At East catch slopes of parking lot.

Surface area disturbed (square feet and acres)*: 28,850 sf 0.66 ac Volume of excavation/fill (yd^3): 334 cut. 299 fill (35 net cut)

Completed By: Humber Design Group

Date: 5/14/2019

*Construction activities disturbing between 1 and 5 acres are automatically covered under the Oregon (DEQ) Department of Environmental Quality (NPDES) National Pollutant Discharge Elimination System Stormwater Discharge General Permit No.1200-CN. This relieves many applicants from also having to apply for a DEQ permit. Activities disturbing over 5 acres are not eligible for automatic coverage and are subject to additional permitting requirements by DEQ under the 1200-C program. Please ask the planning office for a copy of the "GENERAL PERMIT NATIONAL POLLUTANT DISCHARGE ELIMINATION SYSTEM STORMWATER DISCHARGE PERMIT" provisions for more information on projects qualifying for automatic coverage.

SUBMITTAL REQUIREMENTS

This worksheet has been put together to assist you in addressing approval criteria. Additional information is required to submit an application. This includes a General Application Form, deeds, site plan, service provider forms and title report. Please reference the Hillside Development Permit Handout for a list of submittal and site plan requirements.

GEOTECHNICAL ANALYSIS

A Hillside Development Permit may be approved by the County only after the applicant provides one of the following. Please check the applicable box.

☐ **Topographic information is enclosed showing the proposed development to be on land with average slopes less than 25 percent, and located more than 200 feet from a known landslide, and that no cuts or fills in excess of 6 feet in height are planned. High groundwater conditions shall be assumed unless documentation is available, demonstrating otherwise; or**

☒ **A geotechnical report prepared by a Certified Engineering Geologist or Geotechnical Engineer is attached certifying that the site is suitable for the proposed development. The report includes any specific investigations required by the County and recommendations for any further work or changes in proposed work which may be necessary to ensure reasonable safety from earth movement hazards; or**

☒ **An HDP Form– 1 completed, signed and certified by a Certified Engineering Geologist or Geotechnical Engineer with his/her stamp and signature affixed has been prepared indicating that the site is suitable for the proposed development.**

NOTE: If the HDP Form– 1 indicates a need for further investigation, or if the Director requires further study based upon in-formation contained in the HDP Form– 1, a geotechnical report as specified by the Director shall be prepared and submitted.

HDP APPROVAL STANDARDS

County approval of development plans must be based upon findings that the proposal adequately addresses the standards listed below. Some of the standards can be satisfied by checking the corresponding box. By checking a box, you are confirming that the statement applies to your project.

1. **Fill materials, compaction methods and density specifications shall be indicated. Fill areas intended to support structures shall be identified on the plan.**

   ☒ The fill materials, compaction methods and density specifications are included on the site plan or are described below. Fill areas intended to support structures are identified on the plan.

   ☐ There is no fill included in the proposed project.

   Structural fill should consist of imported granular material or approved on site native soil. Fill should be compacted to at least 95% of the maximum dry density as determined by ASTM D-698 or until well keyed. Fine grained fill soils should be compacted with a sheepsfoot while granular fill should be compacted with a vibratory roller or vibratory plate compactors.

2. **Cut and fill slopes shall not be steeper than 3(H):1(V) (i.e. 33%) unless a geological and/or engineering analysis certifies that the steep slopes are safe and erosion control measures are specified.**

   ☐ Cut or fill slopes steeper than 33% have been certified as safe in the attached geological and/or engineering analysis. Appropriate erosion control measures are also specified in the analysis.

   ☒ There are no cut or fill slopes steeper than 33%.

3. **Cuts and fills will not endanger or disturb adjoining property.**

   ☒ A Geotechnical Reconnaissance (HDP Form 1) or geotechnical report has been prepared confirming that cut or fills will not endanger or disturb adjoining property.

   ☐ Cuts and fills will not endanger or disturb adjoining property for the following reasons:

   Note: This issue is specifically addressed in the HDP Form 1 and you can rely upon the response by the Certified Engineering Geologist or Geotechnical Engineer that completed the form. A geotechnical report may or may not address the issue. If you need to prepare a response, please make sure to address any earthwork that is to occur close to a property line or storm run-off that will discharge off the property.

4. **The proposed drainage system will have adequate capacity to bypass through the development the existing upstream flow from a storm of 10-year design frequency;**

   ☐ A County Stormwater Certificate completed by an Oregon Registered Professional Engineer demonstrates that this standard has been satisfied *(Note: A Certificate must be submitted for projects involving more than 500 square feet of impervious surfaces).*

☒ There is no existing upstream flow of run-off.

5. **Fills shall not encroach on natural watercourses or constructed channels unless measures are approved which will adequately handle the displaced stream flow for a storm of 10-year design frequency;**

☐ Fill will encroach on a natural watercourse or constructed channel as shown on the site plan. As illustrated on the plan, and confirmed with the enclosed Stormwater Certificate, adequate measures will be put in place to handle the stream flow for a storm of 10-year design frequency. (*Note: A separate Flood Hazard Permit is required).*

☒ A site plan has been provided demonstrating that fill work will not encroach on natural watercourses or constructed channels.

6. **On sites within the Tualatin River Drainage Basin, specific stormwater and erosion control standards apply. The Basin includes unincorporated rural areas west of Skyline Boulevard.**

☒ The development site is outside of the Tualatin River Drainage Basin (skip to standard #7).

☐ The site is within the Tualatin River Drainage Basin and:

- Measures for controlling erosion and stormwater have been designed to perform as prescribed by the currently adopted edition of the City of Portland Erosion and Sediment Control and Stormwater Management Manuals; and
- The stormwater system has been designed to manage runoff onsite to the maximum extent possible; and
- Land-disturbing activities are at least a 100-foot from the top of the bank of a stream or ordinary high watermark (line of vegetation) of a water body, or a mitigation plan consistent with OAR 340 is enclosed for alterations within the buffer area.

(Note: For the mitigation plan, the County utilizes vegetated corridor provisions contained in Clean Water Services Design and Construction Standards manual. A copy of the manual is available on their website at http://www.cleanwaterservices.org. On slopes less than 25 percent, land disturbing activities can be approved to within 50 feet of a water body provided at least 80 percent of the intervening area is planted with native trees, shrubs, and groundcover that will achieve at least 50% canopy coverage at maturity. Mitigation must occur at a minimum 1:1 ratio to disturbed areas. If your site does not fall within these parameters, other options may exist which you can discuss with our staff.)

7. **Stripping of vegetation, grading, or other soil disturbance shall be done in a manner which will minimize soil erosion, stabilize the soil as quickly as practicable, and expose the smallest practical area at any one time during construction. Please explain how the proposed development meets this standard.**

With the small size of project and its specific construction areas it is anticipated that they will be exposed, stabilized, constructed and/or landscaped as individual treatments which will meet this standard using typical erosion and sediment control measures as dictated in C1.0 & C3.0

8. **Development Plans shall minimize cut or fill operations and ensure conformity with topography so as to create the least erosion potential and adequately accommodate the volume and velocity of surface runoff. Please explain how the proposed development meets this standard.**

The design submitted shows a relatively unchanged topography and the drainage pattern will be maintained across the site. The added paths will remain very close to existing grades in most cases and will have 3" of earth excavation for 3" of gravel path material. The existing Columbia Avenue and private parking lot will have minimal excavation to clear scrub. Refer to sheet C1.0.

9. **Temporary vegetation and/or mulching shall be used to protect exposed critical areas during development.** *(Note: Critical areas are typically soils that if exposed are likely to erode into drainageways or onto roads or nearby properties.)*

☐ The attached erosion control plan includes the use of temporary vegetation and/or mulch to protect exposed soils.

☒ There will be no exposed critical areas. Please explain

The design submitted shows a relatively unchanged topography. Areas of exposed soil intended as a path of travel (vehicular or pedestrian) will receive new improved gravel surfacing. Areas of exposed soil for landscaping will have a finish grade of 5:1 max. and will be immediately reseeded and mulched following final grading to prevent erosion. Refer to sheet C1.0 & C3.0.

10. **Whenever feasible, natural vegetation shall be retained, protected, and supplemented. Please explain how the proposed development meets this standard.**

The proposed design seeks to retain as much of the existing vegetation as practicable. As indicated on sheet L0.01, only a select number of plantings are to be removed, while the majority of existing trees and screenings are to remain and be protected. Native ferns, shrubs and groundcover will be planted throughout the site to help further supplement existing trees and new planting areas are provided in previously disturbed areas. Refer to sheet L0.01 and L2.01.

Also, check one of the following:

☒ The site plan provided shows that a 100-foot undisturbed buffer of natural vegetation will be retained from the top of the bank of a stream, or from the ordinary high watermark (line of vegetation) of a water body, or wetland; or

☐ Development will encroach within the 100 foot buffer. A mitigation plan is enclosed utilizing erosion control and stormwater measures prescribed by the currently adopted edition of the City of Portland Erosion and Sediment Control and Stormwater Management Manuals. The plan further meets surface water quality equivalent to those established for the Tualatin River Drainage Basin in OAR 340. *(Note: See note under item #6 regarding mitigation plan requirements).*

11. **Permanent plantings and any required structural erosion control and drainage measures will be installed as soon as practical. Please explain how the proposed development meets this standard.**

Contractor to install all erosion control measures as noted prior to breaking ground. Proposed utility plan does not require any drainage/erosion control measures as designed. There are no existing or proposed concentrated flows to manage as such. Permanent plantings are scheduled to be installed as part of the general construction contract, and therefore shall be in place prior to overall completion of project construction.

12. **Provisions shall be made to effectively accommodate increased runoff caused by altered soil and surface conditions during and after development. The rate of surface water runoff shall be structurally retarded where necessary. Please explain how the proposed development meets this standard.**

On-site, increased runoff is not anticipated as new impervious surfaces will not be constructed. Both during and following construction the runoff shall be decreased with the addition of gravel at surface in back yard, loading zone, and parking lot. In the public ROW, the existing roadway consists of heavily compacted gravel, so an increase in runoff is not anticipated with the proposed new asphalt paving. Refer to sheet C1.0 & C3.0.

13. **Sediment in the runoff water shall be trapped by use of debris basins, silt traps, or other measures until the disturbed area is stabilized.**

☐ The site plan provided includes debris basins, silt traps, or other measures (specify:_____) which will be installed and maintained until the disturbed areas are stabilized.

☒ The development will not generate sediment laden run-off to warrant the installation of these measures. Please explain:

The site will be wrapped in erosion control measures such as sediment fencing and silt trap for the single existing area drain. The majority of site is and will remain gravel or lawn and thus infiltrates directly, creating no run off to become sediment laden. Refer to sheet C1.0 & C3.0.

14. **Provisions shall be made to prevent surface water from damaging the cut face of excavations or the sloping surface of fills by installation of temporary or permanent drainage across or above such areas, or by other suitable stabilization measures such as mulching or seeding. Please explain how the proposed development meets this standard.**

While there are no existing drainage ditches to line or seed, all cut/fill slopes outside of building envelope shall be grassed and or landscaped as soon as practical to protected from erosion as directed on sheet C1.0 & C3.0.

15. **All drainage provisions shall be designed to adequately carry existing and potential surface runoff to suitable drainageways such as storm drains, natural watercourses, drainage swales, or an approved drywell system.**

☒ Drainage improvements shown on the site plan have been designed to carry existing and potential surface runoff to the following drainageway: Subsurface infiltration for the public ROW

☒ No drainage improvements are associated with the development.

16. **Drainage swales used to divert surface water shall be vegetated or protected to minimize erosion.**

☐ Drainage swales are being used and will be protected to minimize potential erosion. Method of protection:

☒ No drainage swales will be installed.

17. **Erosion and sediment control devices shall be employed where necessary to prevent polluting discharges from occurring. These may include, but are not limited to:**

- **Energy absorbing devices to reduce runoff water velocity;**
- **Sedimentation controls such as sediment or debris basins. Any trapped materials shall be removed to an approved disposal site on an approved schedule;**
- **Dispersal of water runoff from developed areas over large undisturbed areas.**

☐ Erosion control devices of this type are being employed to prevent pollution discharges as shown on the site plan.

☒ No devices are needed to prevent pollution discharges from occurring. Please explain:

The existing and proposed stormwater for the site shall infiltrate through either pervious surfaces or infiltration devices. The existing roof drains to an existing soakage trench and the proposed roof drains connect to a new drywell.

18. **Disposed spoil material or stock-piled topsoil shall be prevented from eroding into streams or drainageways by applying mulch or other protective covering; or by location at a sufficient distance from streams or drainageways; or by other sediment reduction measures.**

☒ As noted on the plan, stockpiled spoils or topsoil will be covered and are located such that they will not erode into nearby streams or drainages.

☐ Spoil material or topsoil will be removed as it is excavated and will not be stored on-site.

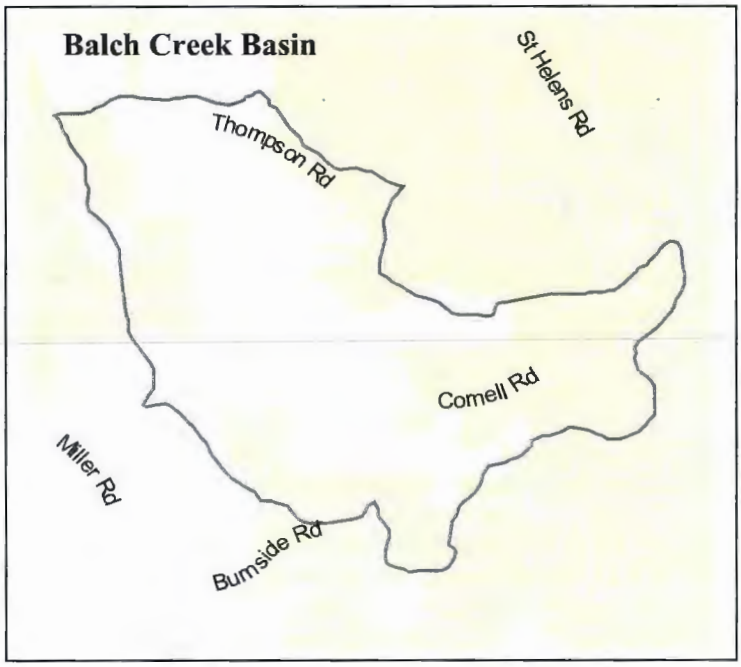
19. **Such non-erosion pollution associated with construction such as pesticides, fertilizers, petrochemicals, solid wastes, construction chemicals, or wastewaters shall be prevented from leaving the construction site through proper handling, disposal, continuous site monitoring and clean-up activities. Please explain how the proposed development will meet this standard.**

A site specific Storm Water Pollution Prevention Plan will be created prior to the start of construction. The SWPPP will outline the site perimeter silt containment, disposal, practices and monitoring schedule. All construction waste will be disposed of in a dumpster or hauled off the site in a truck. A Spill Prevention & Control Plan will be in place to manage any on-site contamination.

20. **On sites within the Balch Creek Drainage Basin, land disturbing activities are limited to the period between May first and October first of any year. All permanent vegetation or a winter cover crop shall be seeded or planted by October first of the same year the development was begun; all soil not covered by buildings or other impervious surfaces must be completely vegetated by December first of the same year the development was begun. The following is a map depicting the boundaries of the Balch Creek Drainage basin.**

☐ The property resides within the Balch Creek Drainage basin. This application has been tailored with the understanding that land disturbing work will be limited to the period between May 1st and October 1st and that cover crops must be established within this timeframe.

☒ The property is not located within the Balch Creek Drainage Basin.



GRI
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December 5, 2018

6060 GEOTECHNICAL RPT

HSF, LLC
c/o Emerick Architects P.C.
321 SW 4th Avenue, Suite 200
Portland, OR 97204

Attention: Keith Daily

SUBJECT: Geotechnical Engineering Investigation
View Point Inn
40301 E Larch Mountain Road
Corbett, Oregon

At your request, GRI completed a geotechnical investigation for the proposed View Point Inn renovations and basement construction in Corbett, Oregon. The Vicinity Map, Figure 1, shows the general location of the site. The purpose of our investigation was to evaluate subsurface conditions at the site and develop geotechnical conclusions and recommendations for design and construction of the renovation. The investigation included a review of available geologic information for the site, subsurface explorations and geologic reconnaissance, laboratory testing, and engineering analyses. This report describes the work accomplished and provides our conclusions and recommendations regarding earthwork and site preparation, foundation and floor support, lateral earth pressures, drainage, utilities, and seismic design considerations.

PROJECT DESCRIPTION

The configuration of the existing building is shown on the Site Plan, Figure 2. The two-story building has a partial basement within the eastern portion of the structure, and the remainder of the structure is on-grade at about elevation 958.6 ft (North American Vertical Datum of 1988 [NAVD 88]). All elevations in this report reference NAVD 88, unless otherwise noted. We understand the proposed renovations to the View Point Inn will include adding a full-depth basement beneath the existing structure to finish floor elevation 948.6 ft. This will require underpinning or otherwise supporting the existing structure and excavating about 10 ft below the existing floor. The existing partial basement beneath the eastern portion of the structure will be removed and replaced as part of the new basement construction. The detached shed to the north will be rebuilt and the gravel parking lot east of the structure may be paved with asphalt concrete (AC) pavement.

SITE DESCRIPTION

Surface Conditions and Topography

The site is bordered by E Larch Mountain Road to the south, a near-vertical cut slope for the Historic Columbia River Highway (HCRH) to the west, and properties owned by the US Forest Service to the north and east. NE Columbia Avenue splits the property into two parcels with the larger lot (with the building) to the west of NE Columbia Avenue and a gravel parking lot to the east of NE Columbia Avenue. The HCRH cut slope was likely completed during original highway construction around 1915.

EXHIBIT
I.12

Review of available topographic maps and lidar elevation data indicate most of the site is relatively flat at elevations ranging from about elevation 960 to 945 ft. The site is on a ridge generally oriented north to south where the ground surface slopes steeply down to the west and to the northeast. The ground to the northeast of the site generally slopes down due to a valley of a small unnamed creek. The ground surface slopes gradually upward south of the site and gradually downward north of the site. A near-vertical cut slope about 90 ft high (oriented in a generally north–to-south direction) is present along the western property boundary for the site and the HCRH. The site is surfaced with AC pavement or crushed-rock surfacing, grass and landscaping, and sidewalks.

Geology

The site is located on a ridge that extends north from E Larch Mountain Road along the HCRH to the Vista House located approximately 2,600 ft northeast of the site. Geologic mapping completed for the area by GRI engineering geologists indicates the site is located on the hyaloclastic sandstone member of the Troutdale Formation. The Troutdale Formation is a Pliocene age, typically weak rock consisting of coarse-grained sandstone and conglomerate largely composed of basalt and glassy debris. The glassy debris contain minerals that are variably weathered to a yellow-brown expansive clay (Evarts, 2006). Underlying the Troutdale Formation is the Basalt of Broughton Bluffs. The Basalt of Broughton Bluffs originate from a volcanic center that represent the late Pliocene to Quaternary Boring Volcanic Field (Fleck et al., 2002). The conglomerate member of the Troutdale Formation underlies the Basalt of Broughton Bluffs and consists of weakly cemented, well-rounded, moderately weathered gravels with a thickness greater than 400 ft. The well-rounded pebbles and cobbles include basalt, granite, and quartzite.

Landslide Mapping

The Oregon Department of Geology and Mineral Industries (DOGAMI) is the state agency responsible for geologic hazard mapping for the State of Oregon. DOGAMI indicated in its statewide landslide hazard database (SLIDO) that on the slope immediately adjacent to the west side of the site, there is a dormant, prehistoric (>150 years), deep-seated landslide, referred to as Washougal 103 (Burns and Watzig, 2014). The failure depth estimated by DOGAMI is approximately 139 ft. Additionally, the site is mapped as having a moderate to high landslide susceptibility. Based on the historical landslide data inventory contained in SLIDO, a rockfall event occurred on the HCRH approximately 250 ft north of the site on February 9, 1996. The approximate location of the landslides identified by DOGAMI is shown on Figure 3. It should be noted that the cut slope for the HCRH west of the site is essentially coincident with a portion of the headscarp of the DOGAMI landslide Washougal 103.

In 2006 GRI assisted the Oregon Department of Transportation (ODOT) with a shallow slope repair along the southbound lane of the HCRH about 200 ft northwest of the project boundary.

Site Reconnaissance

A certified engineering geologist from GRI completed a walking geologic reconnaissance of the site on January 15, 2018. GRI conducted the reconnaissance to evaluate surface conditions at the site for obvious indications of potential slope instability.

The structure on the site consists of a two-story, wood-framed building. A grass-covered yard slopes gradually downward about 10° to the west for about 125 ft west of the building. A concrete sidewalk extends about 75 ft west of the building to an empty, concrete-lined pond and fountain. The concrete in the pond contains

cracks and is broken in places. No obvious cracks or patched cracks are visible on the concrete sidewalks of the site. No ground cracks, depressions, exposed soil, or other indications of potential slope instability are visible around the site buildings.

Approximately 125 ft west of the building is an ornamental iron fence founded in a concrete block oriented parallel to the top of the bluff. Mature Douglas fir trees with J-shaped trunks, suggesting shallow soil creep, are growing near the north and south ends of the fence. The fence serves as a boundary from gentle slope east of the fence and grades to an approximately 90-ft-high cut slope located west of the fence. The slope angle beyond the fence increases to about 50 to 65° descending the slope towards the highway. This portion of the slope contains deciduous trees that have been cut several times to maintain the view and grow in dense clumps along the top to the slope. Growing below the trees are ferns, grasses, and blackberry bushes. Exposed soil in shallow channels was observed near the foundation of the iron fence, indicating minor erosion due to stormwater runoff in this area. The steep terrain prevented safe access to evaluate the base of the fence foundation for undermining by erosion of the slope. A qualified rope-access professional would be necessary to safety observe conditions below the fence. Overhanging root wads with exposed soil from erosion, small and shallow ground cracks, and small shallow slumps are present at the top of the steep bluff below the fence. Curvilinear scarps and half-disc shaped topographic negatives suggest potential for previous shallow, non-deep-seated slumps along the top of the slope.

Beyond about 15 to 20 ft west of the fence, the slope angle increases to approximately 65 to 85° along the location of the HCRH cut slope. This cut slope exposes three rock types present below the site. Rock types exposed on the cut slope include, from top to base, sandstone, basalt, and conglomerate.

Sandstone was observed near the upper elevation exposed on the top of the cut slope. The sandstone appeared to consist of moderately weathered to predominately decomposed, medium to thick bedded, medium- to coarse-grained sandstone and cross-bedded, coarse-grained, angular gravel and scattered cobbles. Large mica flakes, obsidian, and yellow cemented clay are present in the sandstone. The sandstone characteristics are consistent with the description of the hyaloclastic member of the Troutdale Formation.

Basalt rock exposed below the sandstone is black to dark gray, medium hard to hard (R3 to R4), fresh to moderately weathered Basalt of Broughton Bluff, which is a unit of the Boring Lavas. The upper portion of the basalt is moderately weathered, columnar basalt. The lower portion of the outcrop is fresh to slightly weathered and has columnar joints with curvilinear faces. Exposed interior fracture faces are discolored by soil and some of these fractures contain soil infilling. Basalt columns range from about 2 to 5 ft across. Relatively wide discontinuity apertures were observed, and several blocks appeared to be overhanging wedges with little support beneath the about 2- to 3-ft blocks.

Underlying the basalt is a conglomerate composed of moderately weathered to predominantly decomposed, rounded gravel, cobbles, and scattered boulders. This rock type is consistent with descriptions of the Troutdale Formation.

Rock debris is present in the catchment ditch at the base of the slope and a thin layer of soil material coats the face of the slope for about 10 to 15 ft up from the bottom of the slope. Vegetation is absent along the bluff west of the site, except for some moss, along the steepest portion of the bluff. Flowing water was observed exiting the slope at the contact of the sandstone and basalt located in an area approximately 10 ft in length at the contact.

Excluding the HCRH cut slope coincident with the headscarp of the large dormant, prehistoric DOGAMI-mapped landslide Washougal 103, obvious indications of large-scale, deep-seated slope instability that could affect the proposed project were not observed. Based on our experience with similar projects, our understanding of local geologic conditions, and our surface reconnaissance observations, in our opinion, the proposed View Point Inn project will not adversely affect the Washougal 103 feature. It should be anticipated that maintenance of the slope along the western property line will be occasionally required.

SUBSURFACE CONDITIONS

General

Subsurface materials and conditions at the site were investigated on January 2, 3, 12, and 15, 2018, with 12 borings, designated B-1 through B-9 and HA-1 through HA-3. Borings B-1 through B-9 were advanced using a truck-mounted drill rig and borings HA-1 and HA-3 were advanced using hand-operated equipment. The borings were advanced to depths of 14.0 to 41.5 ft below the ground surface at the locations shown on Figure 2. Advancement of borings B-1 through B-6 utilized mud-rotary drilling techniques, and advancement of borings B-7 through B-9 utilized hollow-stem auger techniques. Infiltration testing was performed in borings B-8 and B-9 and in hand-augered borings HA-1 and HA-2. The field and laboratory testing programs completed for this project are described in Appendix A. Logs of the borings are provided on Figures 1A through 11A. The terms used to describe the soil and rock encountered in the borings are defined in Tables 1A and 2A, respectively. The symbols used on the logs are defined on the attached legend.

A portable dynamic cone penetrometer (DCP) penetration test, designated DCP-1, was conducted in accordance with ASTM International (ASTM) D6951 at the location of the existing shed to evaluate the in-situ strength of the near-surface soils. The DCP test was advanced to a depth of about 8 ft below the ground surface. The DCP test result is presented on Figure 12A in Appendix A.

Sampling

Disturbed and undisturbed soil samples were typically obtained from borings B-1 through B-6 at 2.5-ft intervals of depth in the upper 15 ft and 5-ft intervals below this depth. Disturbed samples were also obtained at 5-ft intervals in boring B-7, at a depth of 12.5 and 14.5 ft in boring B-8 and B-9, respectively, and at 2-ft intervals in hand-augered borings HA-1 and HA-2. Disturbed soil samples were obtained using a 2-in.-outside-diameter (O.D.) standard split-spoon sampler while conducting Standard Penetration Tests (SPT) in the borings and collected by hand in the hand-augered boring locations. SPTs were conducted by driving the samplers into the soil a distance of 18 in. using a 140-lb hammer dropped 30 in. The number of blows required to drive the standard split-spoon sampler the last 12 in. is known as the Standard Penetration Resistance, or SPT N-value. The SPT N-values provide a measure of the relative density of granular soils and the relative consistency of cohesive soils. Relatively undisturbed soil samples were collected by pushing a 3-in.-O.D. Shelby tube into the undisturbed soil a maximum of 24 in. using the hydraulic ram of the drill rig. The soil in the Shelby tubes was extruded in our laboratory and Torvane measurements were recorded on selected samples.

Soils

For the purpose of discussion, the materials disclosed by the borings have been grouped into the following major units based on their physical characteristics and engineering properties.

1. **Portland Cement Concrete PAVEMENT**
2. **SILT (Fill)**
3. **SILT (Loess)**
4. **Sandy SILT to Silty SAND (Residual Troutdale Formation)**
5. **SANDSTONE (Troutdale Formation)**

The following paragraphs provide a detailed description of the soil units and a discussion of the groundwater conditions at the site.

1. Portland Cement Concrete PAVEMENT. Portions of the ground surface surrounding the existing building are covered with portland cement concrete (PCC) hardscape or pavement. Borings B-3, B-4, and B-6 encountered about 4.0 to 6.5 in. of PCC pavement at the ground surface. A 4- to 6-in.-thick layer of crushed-rock base (CRB) course was encountered below the PCC in borings B-3, B-4, and B-6.

2. SILT (Fill). Silt fill was encountered at the ground surface in borings B-1 and B-2 and hand-augered borings HA-1 and HA-2 and extends to depths of about 2.5 to 5 ft below the ground surface. The silt fill is typically dark brown and contains a trace of fine- to medium-grained sand. SPT N-values of 2 blows/ft at a depth of 2.5 ft in borings B-1 and B-2 indicate the silt fill is very soft to soft at this depth. The natural moisture content of the silt fill ranges from about 32 to 43%.

Deeper fill materials may be present on the east side of the existing building in the location of a previously removed underground storage tank. The type of fill material used to backfill the storage tank excavation is unknown at this time. Fill of unknown properties may be encountered during the course of construction throughout the site; the fill may be unsuitable for use as structural fill or as support for structural loading.

3. SILT (Loess). Silt was encountered beneath the silt fill in borings B-1 and B-2 and in hand-augered borings HA-1 and HA-2; below the CRB course in borings B-3, B-4, and B-6; and at the ground surface in borings B-5 and B-7 through B-9 and hand-augered boring HA-3. The silt extends to depths of between 8 and 12.5 ft in borings B-1 through B-6, B-8, and HA-3, and to the maximum depths of exploration in borings B-7, B-9, HA-1, and HA-2. The silt is typically brown and contains a trace of fine- to coarse-grained sand to sandy. Based on SPT N-values of 2 to 9, and Torvane strength values of 0.10 to 0.30 tsf, the consistency of the silt ranges from very soft to stiff. The natural moisture content of the alluvial silt ranges from 23 to 43%.

Consolidation testing was performed on selected samples of silt obtained from boring B-1 at a depth of 5.5 ft, boring B-2 at a depth of 6.25 ft. Test results indicate the soil is moderately overconsolidated and has a low compressibility in the preconsolidated range of stresses and a moderate compressibility in the normally consolidated range of stresses, see Figures 14A and 15A.

Atterberg limits testing completed for two samples on the silt indicates the silt has a low plasticity, with liquid limits in the range of 28 to 35% and corresponding plasticity indices from non-plastic to 2.

4. Sandy SILT to Silty SAND (Residual Troutdale Formation). Residual soil is soil-like material that develops from the in-place weathering of rock. Residual soil was encountered below the silt in borings B-1 through B-8 and in boring HA-3. The residual soil is typically brown to gray mottled rust and black and consists of sandy silt to silty sand containing gravel and cobbles. SPT N-values of 4 to greater than 50 blows per 6 in. of sampler penetration (practical refusal) indicate the stiffness and relative density of the residual soil generally

increases with depth from medium stiff/loose to medium dense to stiff/hard and dense to very dense. The natural moisture content of the residual soil ranges from about 32 to 66% and is generally higher between depths of about 13 and 20 ft.

Consolidation testing was performed on selected samples of silt obtained from boring B-5 at a depth of 7.75 ft. Test results indicate the soil is moderately overconsolidated and has a low compressibility in the preconsolidated range of stresses and a moderate compressibility in the normally consolidated range of stresses, see Figure 16A.

Atterberg limits testing completed on a sample of residual soil indicate the soil has a liquid limit of 56 and a plasticity index of 13, which indicates the soil has a medium plasticity.

5. SANDSTONE (Troutdale Formation). Sandstone rock was encountered beneath the residual soil in borings B-1 through B-6 and extends to the maximum depth of exploration in these borings, ranging from 35.1 to 41.5 ft. The sandstone rock is gray to brown, predominantly decomposed and extremely soft (R0). SPT N-values of 48 to 50 blows per less than 6 in. of sampler penetration were measured at the time of sampling. The natural moisture content of the predominantly decomposed rock ranges from 34 to 54%.

Groundwater

Our review of U.S. Geological Survey (USGS) groundwater data (Snyder, 2008) suggests the regional groundwater level beneath the site is expected to be more than 200 ft below the ground surface. Higher moisture contents and wet soil conditions between depth of about 13 and 20 ft can be interpreted as the presence of perched groundwater in the residual soil.

Infiltration Testing

On January 3, 2018, two falling-head infiltration tests, designated I-1 and I-2, were conducted each at a depth of about 5 ft. On January 15, 2018, two additional falling-head infiltration tests, designated I-3 and I-4, were conducted at a depth of about 12.5 and 14.5 ft, respectively. Infiltration tests I-1, I-2, I-3, and I-4 were conducted in borings HA-1, HA-2, B-8, and B-9, respectively, at the approximate locations shown on Figure 2. The infiltration tests were completed in general accordance with the City of Portland's 2016 Stormwater Management Manual (SMM). The unfactored, field-measured infiltration rates recorded at specific depths within a specific soil unit are tabulated below.

Test Location	Depth of Infiltration Test, ft	Average Infiltration Rate, in./hour	Soil Classification
I-1 (HA-1)	5.0	0.0	SILT
I-2 (HA-2)	5.0	0.0	SILT
I-3 (B-8)	12.5	4.0	Sandy SILT to Silty SAND
I-4 (B-9)	14.5	3.5	Sandy SILT

CONCLUSIONS AND RECOMMENDATIONS

General

The subsurface explorations disclosed that the site is mantled with up to about 5 ft of silt fill underlain by silt, residual soil, and predominantly decomposed sandstone rock. The fine-grained silt soil that mantles the site is sensitive to moisture and easily disturbed by construction activities. Site development during wet conditions will require careful working procedures to avoid disturbing and softening the subgrade. In our

opinion, the primary geotechnical considerations with respect to design and construction of the View Point Inn renovation are foundation construction and the excavation of temporary slopes for the basement.

The following sections discuss our recommendations in more detail.

Site Preparation and Grading

We anticipate that minor amounts of grading will be required for areas surrounding the proposed building renovations for this project.

Stripping within the limits of walkways and pavements surrounding the proposed existing building improvements should include removal of any existing hardscape, pavements, vegetation, and surface organics. Existing vaults, basement areas, or other existing below-grade structures should be backfilled with control density fill (CDF) or crushed rock. To minimize disturbance to fine-grained subgrade soils, we recommend using hydraulic excavators equipped with smooth cutting edges for site stripping and excavation in areas where these materials are present. Excavations made during demolition to remove existing improvements should be backfilled with structural fill.

Due to the moisture-sensitive nature of the silty soils that mantle the site, site preparation and earthwork phases of this project will be accomplished most efficiently during the dry summer months. However, if construction is to proceed during the wet months of the year, or if wet ground conditions exist, we recommend making all excavations using large hydraulic excavators (backhoes) equipped with smooth cutting edges, in lieu of scrapers and/or bulldozers, to prevent softening of the subgrade soils. Also, the contractor should plan the earthwork operations such that no construction equipment, i.e., bulldozers, dump trucks, etc., traffic the exposed silty soils. If the subgrade is disturbed during construction, soft disturbed soils should be overexcavated to firm soil and backfilled with clean, granular materials.

In our experience, granular haul roads and work pads generally require a minimum of 18 to 24 in. of relatively clean, fragmental rock to support heavy construction traffic. If the subgrade is particularly soft, it may be advisable to place a woven separation fabric, such as Mirafi 600X (or equivalent), on the exposed subgrade prior to placement and compaction of the granular work pad. The need for haul road can be reduced if work is accomplished during the driest months of the year.

Structural Fill

We anticipate the building's new basement will be excavated to finish grade, and minor amounts of structural fill will be placed for this project. The following paragraphs provide recommendations for structural fill placed for the project, if necessary.

The existing fill and silt that is excavated for the proposed basement footprint and that is free of organics and other unsuitable materials can be re-used as structural fill for general grading. Due to the moisture content of the near-surface soils, it may be difficult and time-consuming to moisture-condition these soils for compaction. If the contractor elects to use imported fill, the fill materials should be free of organics and other unsuitable material and have a maximum size of up to about 4 in.

The natural moisture content of the on-site soils will likely exceed the optimum moisture content throughout most of the year; therefore, some aeration and drying will be required to meet the requirements for proper compaction. The required drying can best be accomplished during dry weather by spreading the material

in thin lifts and disking. Fine-grained soils used as structural fill should be placed in 9-in.-thick lifts (loose) and compacted with segmented-pad or sheepsfoot rollers to 95% of the maximum dry density as determined by ASTM D698. If fine-grained fill soils are compacted at a moisture content that is higher than recommended, the specified densities cannot be achieved, and the fill material will be relatively weak and compressible. Flooding or jetting the backfill with water to achieve the recommended compaction should not be permitted.

On-site, fine-grained soils that are free of debris may be used as fill in landscaped areas. These materials should be placed at about 90% of the maximum dry density as determined by ASTM D698. The moisture content of soils placed in landscaped areas is not as critical, provided that construction equipment can effectively handle the materials.

Seeps or springs that emerge within the basement excavation temporary slopes may require drainage provisions depending on the actual conditions observed during construction. These provisions could include French drains, drainage blankets, and subdrains to collect and remove water. We anticipate the higher moisture contents present between depths of about 13 to 25 ft may indicate the presence of perched groundwater.

Utility Trenches. All backfill placed in utility trench excavations within the limits of the building, sidewalks, hardscape, and paved areas should consist of sand, sand and gravel, or crushed rock with a maximum size of up to 2 in. and with not more than 5% passing the No. 200 sieve (washed analysis). In our opinion, the granular backfill should be placed in lifts and compacted using vibratory plate compactors or tamping units to at least 95% of the maximum density as determined by ASTM D698. Flooding or jetting the backfilled trenches with water to achieve the recommended compaction should not be permitted.

Foundation Support

General. Maximum column and wall loads will be less than 20 kips and 2 kips/ft, respectively. Subsurface explorations disclosed up to about 5 ft of fill across the site. However, local areas of deeper uncontrolled fill may be encountered during excavation for the proposed basement. In our opinion, the uncontrolled fill is not suitable for support of foundations and should be removed from beneath the slab and footings if encountered at basement subgrade. We recommend all footing excavations and subgrades be evaluated by a member of GRI's geotechnical engineering staff as the work progresses.

Shallow Foundations. In our opinion, foundation support for the building renovation can be provided by conventional slab-on-grade and continuous spread footings founded in the underlying silt, residual soil or predominantly decomposed sandstone rock. Wall and spread footings established in these materials can be designed to impose an allowable bearing pressure of 2,500 psf. The allowable bearing pressures apply to the total of dead loads plus permanently applied live loads and can be increased by one-third for the total of all loads: dead, live, and transient (wind or seismic). The minimum width of any footing should not be less than 18 in. for continuous wall footings and 24 in. for spread footings. Footings should be embedded at least 24 in. below the lowest adjacent exterior grade.

Horizontal shear forces can be resisted partially or completely by frictional forces developed between the base of footings and the underlying soil. The total shearing resistance between the footing and the soil or rock should be taken as the normal force, i.e., the sum of all vertical forces (dead load plus real live load), times the coefficient of friction between the soil or rock and the base of the footing. We recommend using

an ultimate coefficient of friction of 0.35 for footings founded on granular structural fill or firm native soil. If additional lateral resistance is required, passive earth pressures from embedded footings or walls may be computed using an equivalent fluid pressure of 300 pcf.

Floor Support

In our opinion, perched groundwater levels during the wet, winter season may be expected to rise to near the basement floor in the area of the proposed building renovation. Therefore, we recommend that the below-grade portions of the building be provided with a subdrainage system to reduce hydrostatic pressure and the risk of groundwater entering through embedded walls and floor slabs. In our opinion, dissipation of the hydrostatic pressures can be achieved by the installation of a suitable subdrain system.

The essential elements of the recommended subdrainage system are shown on Figure 4. The figure shows peripheral subdrains to drain embedded walls and an interior granular drainage blanket beneath the concrete floor slab which is drained by a system of subslab drainage pipes. The subslab drainage pipes should be placed on a center-to-center spacing of 20 ft or less and sloped to drain. All groundwater should be drained by gravity or pumped from sump pits into the storm sewer system. If the water is pumped, it may be prudent to also provide an auxiliary pump and an emergency power supply to prevent flooding in the event of a loss of power. The upper 10 in. of the granular material should serve as a drainage blanket and should consist of angular, open-graded, crushed rock of up to about 1-in. size with not more than 2% passing the No. 200 sieve (washed analysis); 3/4- to 1/4-in. crushed rock is commonly used for this purpose. This material should be installed in a single lift and compacted until well-keyed using a minimum of four passes with a medium- to heavy-weight vibratory roller. To facilitate compaction of the drainage blanket and limit contamination from construction activities prior to placing the concrete slab, it may be desirable to replace the upper 2 in. of the drainage blanket material with 3/4-in.-minus crushed rock having less than 5% passing the No. 200 sieve (washed analysis).

We recommend a vapor-retarding membrane be installed beneath slab-on-grade floors. The membrane should be installed in accordance with the manufacturer's recommendations.

Slope Setback Criteria

The approximate 90-ft-high rock face slope to the west of the property was cut as part of the roadway construction in 1915 and is not anticipated to be significantly affected by the proposed View Point Inn renovations due to the setback of the existing structure. This slope is considered moderately stable, however, it should be anticipated ongoing maintenance will be required to mitigate surficial sloughing that may occur during periods of wet weather.

Excavations and Temporary Shoring

The method of excavation and the design of trench support or shoring are the responsibility of the contractor and subject to applicable local, state, and federal safety regulations, including the current Occupational Safety and Health Administration (OSHA) excavation and trench safety standards. The means, methods, and sequencing of construction operations and site safety are also the responsibility of the contractor. The information provided below is for use by the owner and engineer and should not be interpreted to mean that GRI is assuming responsibility for the contractor's actions or site safety.

We anticipate excavations required for construction of the new basement may extend up to 12 ft below existing site grades. The method of excavation and groundwater management system are the responsibility of the contractor. We recommend the contractor submit for review an excavation and dewatering plan prepared by a professional engineer. The information provided below is for use by the owner and should not be interpreted to mean that GRI is assuming responsibility for the contractor/owner's actions, site safety, or design.

We anticipate conventional excavation equipment can be used to complete the excavations. Based on the proposed excavation depth we anticipate the excavation may encounter groundwater seepage. We anticipate perched groundwater seepage, if encountered can be managed with a system of sump pumps.

We anticipate temporary excavations up to about 10 ft deep can be excavated to a maximum inclination of about 1H:1V (Horizontal to Vertical), and excavations greater than 10 ft deep can be excavated to a maximum inclination of about 1.5H:1V, assuming the slope is dry, is in firm material, and does not have existing improvements or surcharge loading within a horizontal distance from the crest of the slope equal to the height of the slope. Some minor amounts of sloughing, slumping, or running of temporary slopes should be anticipated during and shortly following excavation. If significant seepage or running soil conditions are encountered, it may be necessary to place a blanket of clean, granular fill material against the face of the slope to control these conditions. The contractor should be aware that all excavation and shoring should conform to the requirements specified in the applicable local, state, and federal safety regulations, such as OSHA Health and Safety Standards for Excavations, 29 CFR Part 1926, or successor regulations.

Other measures that should be implemented to reduce the risk of localized failures of temporary slopes include: 1) using plastic sheeting to protect the exposed cut slopes from surface erosion; 2) providing positive drainage away from the top and bottom of the cut slopes; 3) constructing and backfilling walls as soon as practical after completing the excavation; and 4) periodically monitoring the area around the top of the excavation for evidence of ground cracking. It must be emphasized that following these recommendations will not guarantee that sloughing or movement of the temporary cut slopes will not occur; however, the measures should serve to reduce the risk of a major slope failure. Blocks of ground and/or localized slumps may tend to move into the excavation during construction.

In areas where the recommended slope inclination cannot be achieved due to existing or new improvements, the contractor must select an excavation support system that limits the risk of lateral movement and loss of ground and protects adjacent pavements, utilities, and buildings. Potential methods of excavation support include shields, sheet piles, tied-back soldier piles, and plates with hydraulic braces. Temporary lateral support for excavations should be designed to resist lateral earth pressures, hydrostatic pressures, and surcharge effects from traffic, equipment, materials, trench spoils, and adjacent buildings. Lateral earth pressure criteria for temporary braced shoring design can be designed on the basis of a rectangular pressure distribution based on a pressure of 25H (psf), where H is the height of the excavation. Additional lateral earth pressures due to surcharge loads, such as construction equipment, stockpiled materials, and adjacent structures, can be estimated using the criteria provided on Figure 5.

Design Lateral Earth Pressures

Design lateral earth pressures for retaining walls depend on the type of construction, i.e., the ability of the wall to yield. Possible conditions include a wall that is laterally supported at its base and top and, therefore,

is unable to yield, and a conventional cantilevered or gravity wall that yields by tilting about its base. Yielding and non-yielding walls can be designed on the basis of a hydrostatic pressure based on an equivalent fluid unit weight of 35 and 52 pcf, respectively. These pressures assume the embedded walls support a fully drained horizontal backfill. A minimum uniform vertical surcharge pressure of 250 psf should be added to the soil pressure to account for construction equipment in the backfill area. However, these surcharge values should be evaluated based on specific construction equipment utilized. Additional loading due to surcharge loads should be added in accordance with the criteria shown on Figure 5. Evaluation of seismic-induced lateral pressures, in addition to static pressures, can be designed on the basis of a hydrostatic pressure based on an equivalent fluid weight of 5 and 13 pcf for yielding and non-yielding walls, respectively. The resultant force acts at a point above the base of the wall equal to one-third the wall height.

Passive earth pressures against embedded walls can be computed on the basis of an equivalent fluid having a unit weight of 300 pcf. This design passive earth pressure would be applicable only if the embedded wall is cast neat against undisturbed soil or if backfill for the wall is placed as granular structural fill. This value also assumes the ground surface in front of the wall is horizontal, i.e., does not slope down away from the toe of the wall.

Drainage for embedded walls should be provided by a perforated drain pipe located at the bottom of the wall to drain all groundwater by gravity or pumped from sump pits into the storm sewer system. Wall backfill should consist of clean, granular structural fill material compacted to about 95% of the maximum dry density determined by ASTM D698. To provide adequate drainage, we recommend placing a minimum 2-ft-wide vertical drainage layer against the back of the wall during backfilling (see Figure 5). The drainage layer should consist of clean granular material, such as gravel or crushed rock with not more than about 2% passing the No. 200 sieve (washed analysis). Overcompaction of backfill behind the walls should be avoided. Heavy compactors and large pieces of construction equipment should not operate within 4 ft of any embedded walls.

Seismic Considerations

Based on our review of the 2012 International Building Code (IBC) and 2014 Oregon Structural Specialty Code (OSSC), which incorporates recommendations from American Society of Civil Engineers (ASCE) Document 7-10, Minimum Design Loads for Building and Other Structures (ASCE 7-10), and the results of our subsurface investigation, we recommend using Site Class D to evaluate the seismic design of the structure. The 2012 IBC and ASCE 7-10 seismic hazard levels are based on a Risk-Targeted Maximum Considered Earthquake (MCE_R). The ground motion associated with the probabilistic MCE_R represents a targeted risk level of 1% in 50 years probability of collapse in the direction of maximum horizontal response. In general, these risk-targeted ground motions are developed by applying adjustment factors of directivity and risk coefficients to the 2% probability of exceedance in 50 years, or 2,475-year return period hazard level, ground motion developed from the 2008 USGS probabilistic seismic hazard maps. The risk-targeted probabilistic values are also subject to a deterministic limit. The maximum horizontal direction spectral response accelerations were obtained from the USGS Seismic Design Maps for the coordinates of 45.5327° N latitude and 122.2481° W longitude. The S_S and S_1 parameters identified for the site are 0.78 and 0.34 g, respectively. These bedrock spectral ordinates are adjusted for Site Class with the short- and long-period site coefficients, F_a and F_v, based on subsurface conditions or with a site-specific response analysis. The design-level response spectrum is calculated as two-thirds of the Site Class-adjusted MCE_R-level spectrum.

We recommend using the design-level F_a and F_v coefficients for Site Class D to estimate the ground-surface MCE_R spectrum. The spectra are based on a damping ratio of 5%. The MCE_R and design response spectra parameters are tabulated below.

2014 OSSC SEISMIC DESIGN RECOMMENDATIONS

Seismic Variable	Recommended Value
Site Class	D
MCE_R 0.2-Sec Period Spectral Response Acceleration, S_{MS}	0.93 g
MCE_R 1-Second Period Spectral Response Acceleration, S_{M1}	0.58 g
Design 0.2-Second Period Spectral Response Acceleration, S_{DS}	0.62 g
Design 1-Second Period Spectral Response Acceleration, S_{D1}	0.39 g

Based on our understanding of the subsurface conditions at the site and the regional seismicity, we are of the opinion that the major seismic hazard at this site is ground shaking. The potential for earthquake-induced ground rupture, landslides, liquefaction, settlement, and subsidence is low, and the potential for damage by tsunami and/or seiche at the site is absent.

Pavement Design

We anticipate the majority of any new pavement will consist of AC; however, areas subjected to heavy traffic volumes, such as loading docks, or areas subject to repeated heavy truck traffic, may be paved with PCC. Based on our experience with similar projects and subgrade materials, we recommend the following pavement sections.

	Minimum CRB Thickness, in.	Minimum AC Thickness, in.
Areas Subject to Occasional Heavy Truck Traffic	12	4
Areas Subject to Primarily Automobile Traffic and Parking	8	3

	Minimum CRB Thickness, in.	Minimum PCC Thickness, in.
Areas Subject to Heavy Truck Traffic (trash enclosure area or loading docks)	6	6

A geotextile fabric should be placed on the exposed subgrade prior to placement of the CRB.

The recommended pavement sections should be considered minimum thicknesses, and it should be assumed that some maintenance will be required over the life of the pavement (15 to 20 years). The section is based on the assumption that pavement construction will be accomplished during the dry season and after construction of the building has been completed. If wet-weather pavement construction is considered, it will likely be necessary to increase the thickness of CRB to support construction equipment and protect the subgrade from disturbance. The recommended pavement sections are not intended to support extensive

construction traffic, such as dump trucks and concrete trucks. Pavements subject to construction traffic may require repair.

For the recommended pavement sections, drainage is an essential aspect of pavement performance. We recommend all paved areas be provided with positive drainage to remove surface water and water within the base course. This will be particularly important in cut sections or at low points within the paved areas, such as at catch basins. Effective methods to prevent saturation of the base course materials include providing weep holes in the sidewalls of catch basins, subdrains in conjunction with utility excavations, and separate trench drain systems. To provide quality materials and construction practices, we recommend the pavement work conform to ODOT standards.

Prior to placing base course materials, all pavement subgrades should be evaluated by a member of GRI's geotechnical engineering staff. This evaluation may include a proof roll with a fully loaded 10-cy dump truck. Any soft areas detected during the evaluation should be overexcavated to firm ground and backfilled with compacted structural fill.

On-Site Disposal of Stormwater

The unfactored, field-measured infiltration rates for infiltration tests completed at depths of 5, 12.5, and 14.5 ft are 0, 4.0, and 3.5 in./hour, respectively. Table 2-2 in the 2016 SMM specifies a minimum factor of safety of 2 for infiltration design based on encased, falling head infiltration tests. Therefore, we recommend reducing the field infiltration rates by at least 50% to meet the requirements of the 2016 SMM and account for the reduction in the rate of infiltration over time due to clogging.

DESIGN REVIEW AND CONSTRUCTION SERVICES

We welcome the opportunity to review and discuss construction plans and specifications for this project as they are being developed. In addition, GRI should be retained to review all geotechnical-related portions of the plans and specifications to evaluate whether they are in conformance with the recommendations provided in our report. In addition, to observe compliance with the intent of our recommendations, design concepts, and the plans and specifications, we are of the opinion that all construction operations dealing with earthwork and foundations should be observed by a GRI representative. Our construction-phase services will allow for timely design changes if site conditions are encountered that are different from those described in our report. If we do not have the opportunity to confirm our interpretations, assumptions, and analyses during construction, we cannot be responsible for the application of our recommendations to subsurface conditions that are different from those described in this report.

LIMITATIONS

This report has been prepared to aid the project team in the design of this project. The scope is limited to the specific project and location described herein, and our description of the project represents our understanding of the significant aspects of the project relevant to the earthwork and design and construction of foundations and floor support. In the event that any changes in the design and location of the improvements as outlined in this report are planned, we should be given the opportunity to review the changes and to modify or reaffirm the conclusions and recommendations of this report in writing.

The conclusions and recommendations submitted in this report are based on the data obtained from the borings made at the locations indicated on Figure 2 and from other sources of information discussed in this

report. In the performance of subsurface investigations, specific information is obtained at specific locations at specific times. However, it is acknowledged that variations in soil conditions may exist between exploration locations. This report does not reflect variations that may occur between these explorations. The nature and extent of variation may not become evident until construction. If, during construction, subsurface conditions differ from those encountered in the explorations, we should be advised at once so that we can observe and review these conditions and reconsider our recommendations where necessary.

Submitted for GRI,

Renews 06/2020

A. Wesley Spang, PhD, PE, GE
Principal

George Freitag, CEG
Principal

Alex Baumann, PE
Engineering Staff

References

Burns, W. J., and Watzig, R. J., 2014, Statewide landslide information database for Oregon, release 3.2 (SLIDO-3.2): Oregon Department of Geology and Mineral Industries.

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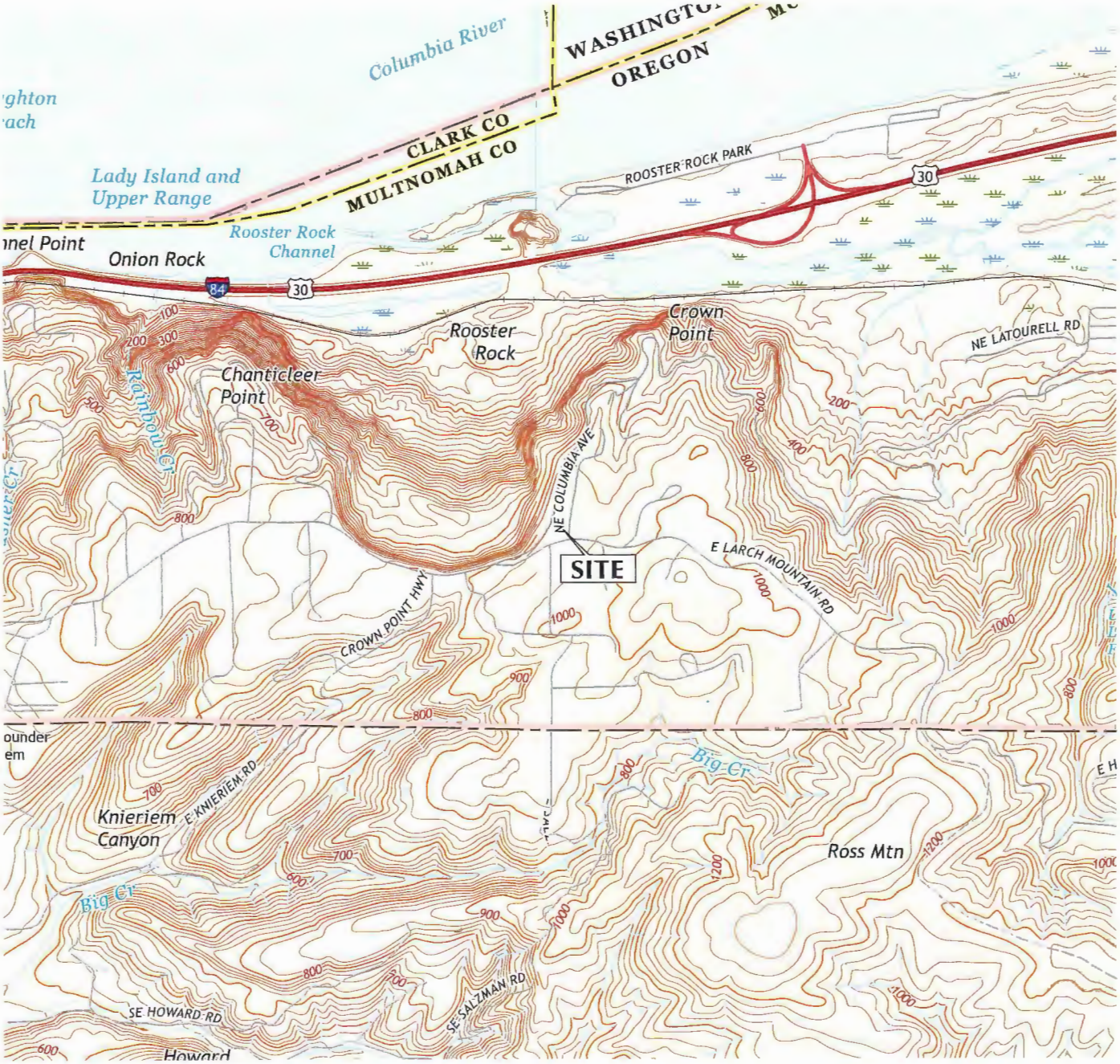
Fleck, R. J., Evarts, R. C., Hagstrum, J. T., and Valentine, M. J., 2002, The Boring Volcanic Field of the Portland, Oregon area— geochronology and neotectonic significance: Geological Society of America Abstracts with Program, vol. 33, no. 5, pp. 33-34.

Snyder, D. T., 2008, Estimated depth to ground water and configuration of the water table in the Portland, Oregon area: U.S. Geological Survey Scientific Investigations Report 2008–5059, 40 p.



USGS TOPOGRAPHIC MAP
WASHOUGAL, WASH. (2017)
BRIDAL VEIL, OREG. (2017)

North

0 1/2 1 MILE

GRI

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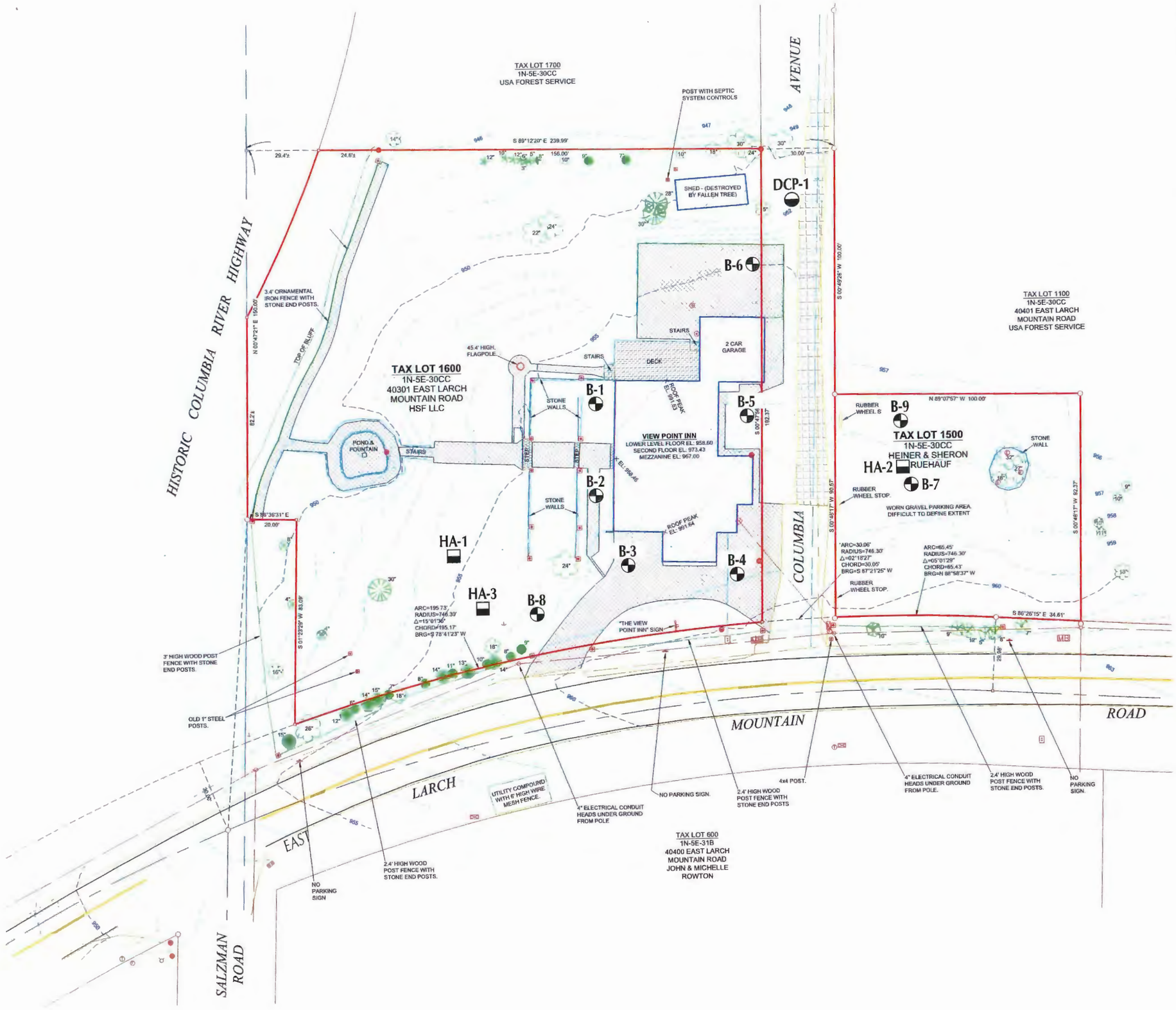
VIEW POINT INN

VICINITY MAP

DEC. 2018 JOB NO. 6060 FIG. 1



BORING COMPLETED BY GRI (JANUARY 2 - 15, 2018)

HAND-AUGERED BORING COMPLETED BY GRI (JANUARY 2 - 12, 2018)

DYNAMIC CONE PENETRATION TEST COMPLETED BY GRI (JANUARY 2, 2018)

SITE PLAN FROM FILE BY EMERICK ARCHITECTS, DATED AUGUST 3, 2017

North

0 40 80 FT

GRI

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SITE PLAN

DEC. 2018 JOB NO. 6060 FIG. 2

- Tsv- - Skamania Volcanic Series
- Tpr- - Columbia River Basalt Group, Priest Rapids Member
- Tgn2h- - Columbia River Basalt, Grande Ronde
- Tf- - Columbia River Basalt, Frenchman Springs Member
- Qls- - Landslides
- Qal- - Alluvium
- QTtu- - Troutdale Formation, Upper Member
- QTb- - Boring Lavas

North

0 1,000 2,000 FT

GRI

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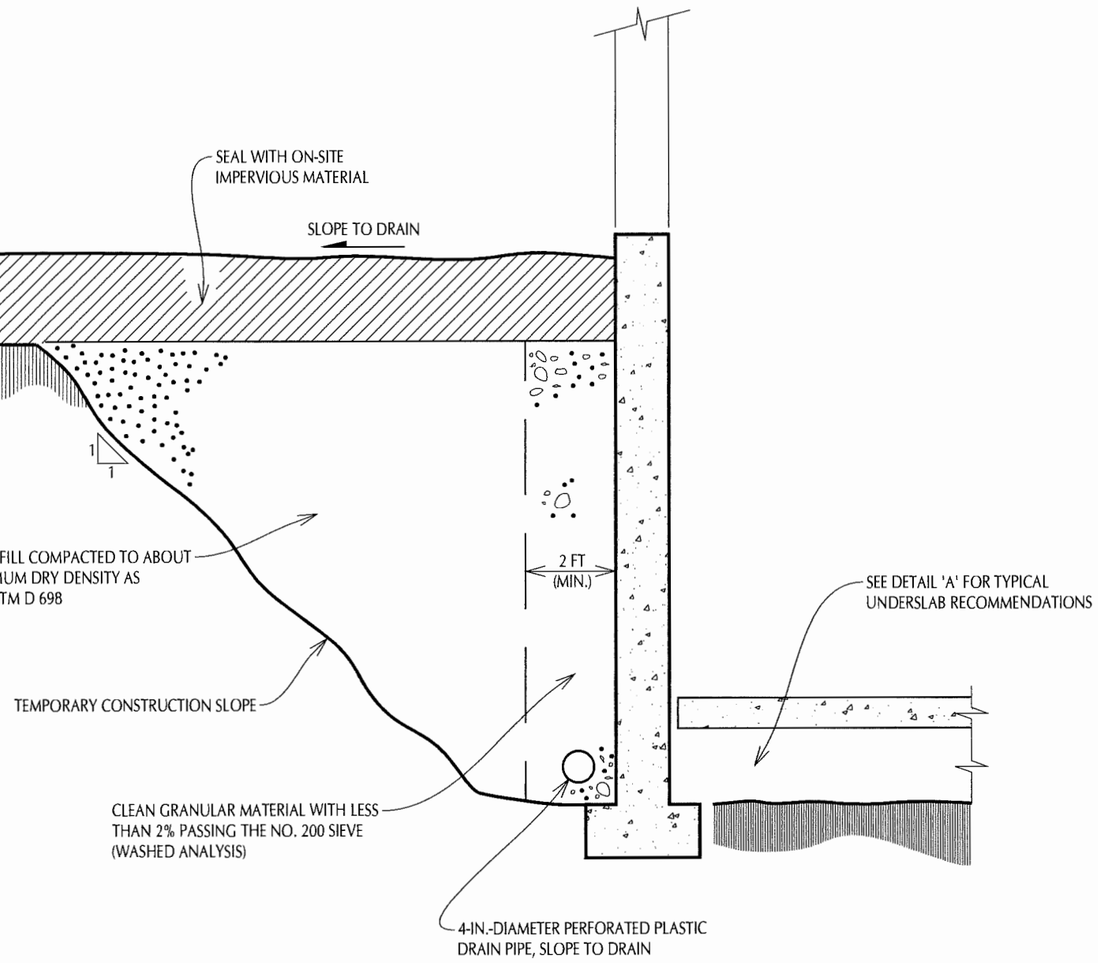
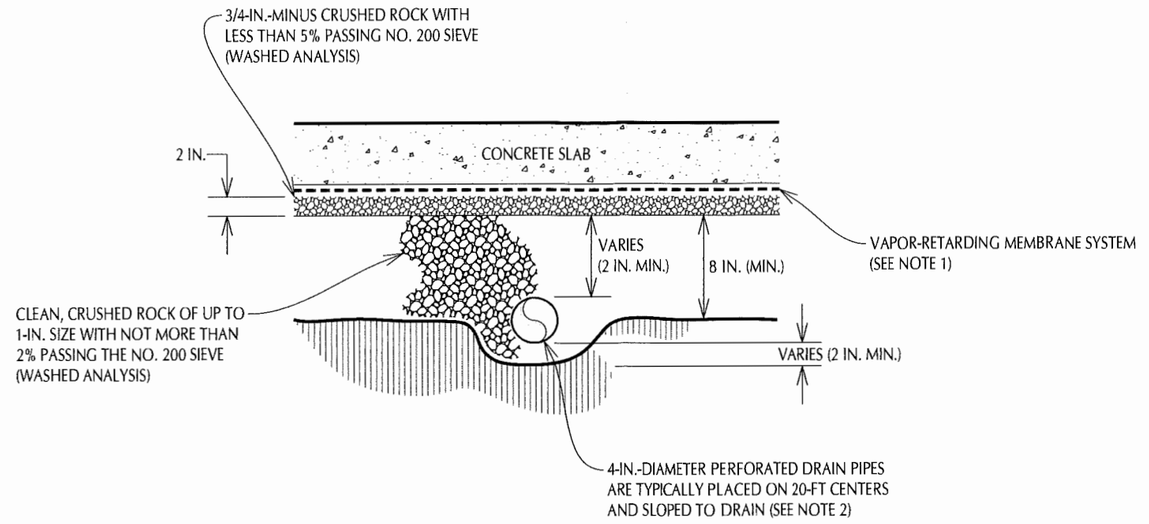
GEOLOGIC MAP

DEC. 2018 JOB NO. 6060 FIG. 3



PERIMETER DRAIN



DETAIL 'A'
NOT TO SCALE

UNDERSLAB DRAIN

NOTES:

1) A VAPOR-RETARDING MEMBRANE SYSTEM IS RECOMMENDED FOR MOISTURE-SENSITIVE AREAS AND SHOULD BE INSTALLED IN ACCORDANCE WITH MANUFACTURER'S RECOMMENDATIONS.

2) INTERNAL 4-IN.-DIAMETER PERFORATED DRAIN PIPES ARE TYPICALLY NOT NECESSARY IN THOSE AREAS WHERE THE FINISH FLOOR WILL BE ABOVE EXISTING SITE GRADES.

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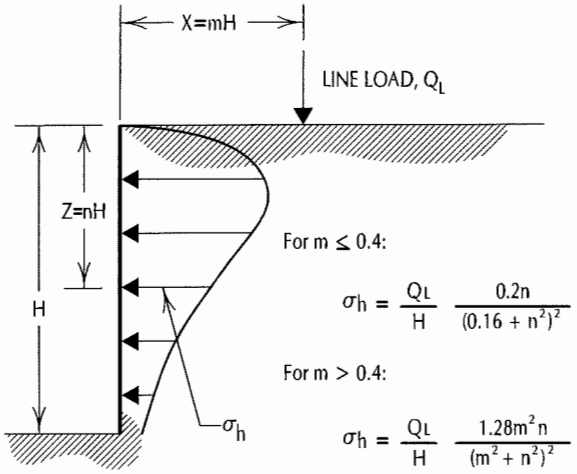
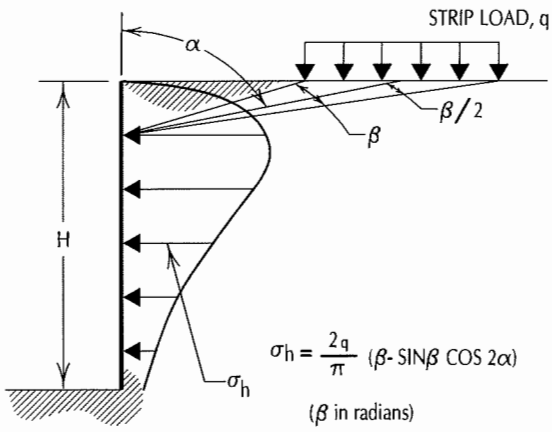
TYPICAL SUBDRAINAGE DETAILS

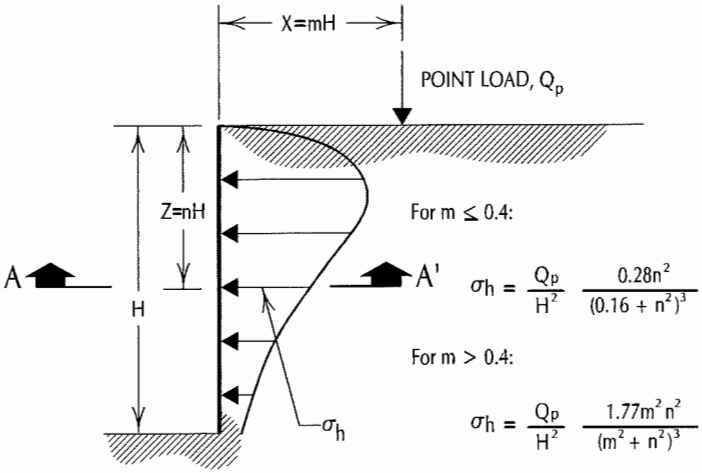
DEC. 2018 JOB NO. 6060 FIG. 4

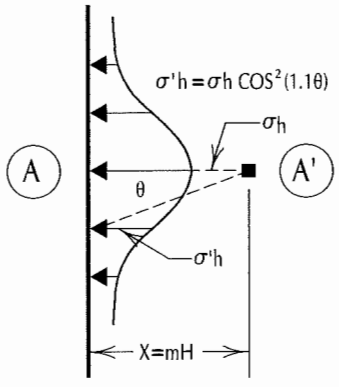


LINE LOAD PARALLEL TO WALL



STRIP LOAD PARALLEL TO WALL





DISTRIBUTION OF HORIZONTAL PRESSURES

VERTICAL POINT LOAD

NOTES:

1. THESE GUIDELINES APPLY TO RIGID WALLS WITH POISSON'S RATIO ASSUMED TO BE 0.5 FOR BACKFILL MATERIALS.

2. LATERAL PRESSURES FROM ANY COMBINATION OF ABOVE LOADS MAY BE DETERMINED BY THE PRINCIPLE OF SUPERPOSITION.

GRI

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VIEW POINT INN

SURCHARGE-INDUCED LATERAL PRESSURE

DEC. 2018 JOB NO. 6060 FIG. 5

APPENDIX A

Field Explorations and Laboratory Testing

APPENDIX A

FIELD EXPLORATIONS AND LABORATORY TESTING

FIELD EXPLORATIONS

Borings

Subsurface materials and conditions at the site were investigated on January 2, 3, 12, and 15, 2018, with nine borings designated B-1 through B-9, and three hand-augered borings designated HA-1 through HA-3. The approximate locations of the explorations are shown on Figure 2. A member of GRI's engineering staff directed the drilling and maintained a log of the materials and conditions disclosed during the course of the work.

Borings B-1 through B-6 were advanced to depths of 14.0 to 41.5 ft using mud-rotary drilling techniques with a truck-mounted drill rig provided and operated by Western States Soil Conservation, Inc., of Hubbard, OR. Borings B-7 through B-8 were advanced using solid-stem auger drilling methods with a trailer-mounted drill rig provided and operated by Greg Vandehey Soil Sampling of Forest Grove, Oregon. Disturbed and undisturbed soil samples were typically obtained from borings B-1 through B-6 at 2.5-ft intervals of depth in the upper 15 ft and 5-ft intervals below this depth. Disturbed samples were also obtained at 5-ft intervals in boring B-7, at depths of 12.5 and 14.5 ft in borings B-8 and B-9, respectively, and at 2 ft intervals in hand-augered borings HA-1 and HA-2. Disturbed soil samples were obtained using a 2-in.-outside-diameter (O.D.) standard split-spoon sampler in the borings and collected by hand in the hand-augers. Standard Penetration Tests (SPT) were conducted by driving the samplers into the soil a distance of 18 in. using a 140-lb hammer dropped 30 in. The number of blows required to drive the standard split-spoon sampler the last 12 in. is known as the Standard Penetration Resistance, or SPT N-value. The SPT N-values provide a measure of relative density of granular soils and the relative consistency of cohesive soils. Relatively undisturbed soil samples were collected by pushing a 3-in.-O.D. Shelby tube into the undisturbed soil a maximum of 24 in. using the hydraulic ram of the drill rig. The soil exposed in the ends of the Shelby tube were examined and classified in the field. The ends of the tube were then sealed with rubber caps. All samples were returned to our laboratory for further examination and physical testing.

Logs of the borings are provided on Figures 1A through 11A. Each log presents a descriptive summary of the various types of materials encountered in the boring and notes the depth at which the materials and/or characteristics of the materials change. To the right of the descriptive summary, the depth to groundwater and the numbers and types of samples are indicated. Farther to the right, SPT N-values are shown graphically, along with Torvane shear strength values, natural moisture contents and fines contents. The terms used to describe the soils and rock encountered in the borings are defined in Tables 1A and 2A. The symbols used on the logs are defined on the attached legend.

Dynamic Cone Penetrometer (DCP) Test

A dynamic cone penetration (DCP) test, designated DCP-1, was advanced to a depth of 8 ft using the Wildcat cone penetrometer manufactured by Triggs Technologies, Inc. The Wildcat cone penetrometer sounding consists of driving a 1.4-in.-diameter cone with a 35-lb weight falling 15 in. The number of blows required to drive the cone 10 cm (approx. 4 in.) is recorded to assess the stiffness characteristics of the underlying

soils. DCP test results are summarized on Figure 12A, which show the blows required to drive the cone tip in 10-cm increments.

LABORATORY TESTING

General

All samples obtained from the field were returned to our laboratory where the physical characteristics of the samples were noted, and the field classifications were modified where necessary. At the time of classification, the natural moisture content of each sample was determined. Additional testing included Torvane shear strength, dry unit weight, Atterberg limits, one-dimensional consolidation. A summary of the laboratory test results is provided in Table 3A. The following paragraphs describe the testing program in more detail.

Natural Moisture Content

Natural moisture content determinations were made in conformance with ASTM D2216. The results are shown on Figures 1A through 11A and are summarized in Table 3A.

Grain-Size Analysis

Washed-Sieve Method. To assist in classification of the soils, samples of known dry weight were washed over a No. 200 sieve. The material retained on the sieve is oven-dried and weighed. The percentage of material passing the No. 200 sieve is then calculated. The results are shown on Figures 1A through 11A and summarized in Table 3A.

Torvane Shear Strength

The approximate undrained shear strength of relatively undisturbed fine-grained soil samples was determined using a Torvane shear device. The Torvane is a hand-held apparatus with vanes which are inserted into the soil. The torque required to fail the soil in shear around the vanes is measured using a calibrated spring. The results of the Torvane shear strength tests are summarized on Figures 1A through 11A.

Dry Unit Weight

The dry unit weight of select undisturbed samples was determined in the laboratory in accordance with ASTM International (ASTM) D2937 by cutting a cylindrical specimen of soil from a Shelby tube sample. The dimensions of the specimen were carefully measured, the volume calculated, and the specimen weighed. After oven-drying, the specimen was re-weighed and the moisture content calculated. The dry unit weight was then computed. The dry unit weights are provided on Figures 1A through 11A and summarized in Table 3A.

Atterberg Limits

Atterberg limits determinations were performed on four selected samples in substantial conformance with ASTM D4318. The test results are shown on the boring logs, Figures 1A through 11A, and the Plasticity Chart, Figure 13A. The results are also summarized in Table 3A.

One-Dimensional Consolidation

Consolidation testing was performed on selected samples in accordance with ASTM D2435 to obtain data on the compressibility characteristics of four relatively undisturbed samples of fine-grained soil. Test results

are summarized on Figures 14A through 16A in the form of a curve showing effective stress versus percent strain. The initial moisture content and unit weight of the sample are provided at the top of the figure.

Table 1A

GUIDELINES FOR CLASSIFICATION OF SOIL

Description of Relative Density for Granular Soil

Relative Density	Standard Penetration Resistance (N-values) blows per ft
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	over 50

Description of Consistency for Fine-Grained (Cohesive) Soils

Consistency	Standard Penetration Resistance (N-values) blows per ft	Torvane or Undrained Shear Strength, tsf
Very Soft	0 - 2	less than 0.125
Soft	2 - 4	0.125 - 0.25
Medium Stiff	4 - 8	0.25 - 0.50
Stiff	8 - 15	0.50 - 1.0
Very Stiff	15 - 30	1.0 - 2.0
Hard	over 30	over 2.0

Grain-Size Classification	Modifier for Subclassification		
Boulders:		Primary Constituent SAND or GRAVEL	Primary Constituent SILT or CLAY
> 12 in.	Adjective	Percentage of Other Material (by weight)	
Cobbles:			
3 - 12 in.	trace:	5 - 15 (sand, gravel)	5 - 15 (sand, gravel)
Gravel:	some:	15 - 30 (sand, gravel)	15 - 30 (sand, gravel)
1/4 - 3/4 in. (fine)	sandy, gravelly:	30 - 50 (sand, gravel)	30 - 50 (sand, gravel)
3/4 - 3 in. (coarse)			
Sand:	trace:	< 5 (silt, clay)	*Relationship of clay and silt determined by plasticity index test*
No. 200 - No. 40 sieve (fine)	some:	5 - 12 (silt, clay)	
No. 40 - No. 10 sieve (medium)	silty, clayey:	12 - 50 (silt, clay)	
No. 10 - No. 4 sieve (coarse)			
Silt/Clay:			
pass No. 200 sieve			

GRI

Table 2A

GUIDELINES FOR CLASSIFICATION OF ROCK

Relative Rock Weathering Scale

Term	Field Identification
Fresh	Crystals are bright. Discontinuities may show some minor surface staining. No discoloration in rock fabric.
Slightly Weathered	Rock mass is generally fresh. Discontinuities are stained and may contain clay. Some discoloration in rock fabric. Decomposition extends up to 1 in. into rock.
Moderately Weathered	Rock mass is decomposed 50% or less. Significant portions of rock show discoloration and weathering effects. Crystals are dull and show visible chemical alteration. Discontinuities are stained and may contain secondary mineral deposits.
Predominantly Decomposed	Rock mass is more than 50% decomposed. Rock can be excavated with geologist's pick. All discontinuities exhibit secondary mineralization. Complete discoloration of rock fabric. Surface of core is friable and usually pitted due to washing out of highly altered minerals by drilling water.
Decomposed	Rock mass is completely decomposed. Original rock "fabric" may be evident. May be reduced to soil with hand pressure.

Relative Rock Hardness Scale

Term	Hardness Designation	Field Identification	Approximate Unconfined Compressive Strength
Extremely Soft	R0	Can be indented with difficulty by thumbnail. May be moldable or friable with finger pressure.	< 100 psi
Very Soft	R1	Crumbles under firm blows with point of geology pick. Can be peeled by a pocket knife and scratched with fingernail.	100 - 1,000 psi
Soft	R2	Can be peeled by a pocket knife with difficulty. Cannot be scratched with fingernail. Shallow indentations made by firm blow of geology pick.	1,000 - 4,000 psi
Medium Hard	R3	Can be scratched by knife or pick. Specimen can be fractured with a single firm blow of hammer/geology pick.	4,000 - 8,000 psi
Hard	R4	Can be scratched with knife or pick only with difficulty. Several hard hammer blows required to fracture specimen.	8,000 - 16,000 psi
Very Hard	R5	Cannot be scratched by knife or sharp pick. Specimen requires many blows of hammer to fracture or chip. Hammer rebounds after impact.	> 16,000 psi

RQD and Rock Quality

Relation of RQD and Rock Quality

RQD (Rock Quality Designation), %	Description of Rock Quality
0 - 25	Very Poor
25 - 50	Poor
50 - 75	Fair
75 - 90	Good
90 - 100	Excellent

Terminology for Planar Surface

Bedding	Joints and Fractures	Spacing
Laminated	Very Close	< 2 in.
Thin	Close	2 in. – 12 in.
Medium	Moderately Close	12 in. – 36 in.
Thick	Wide	36 in. – 10 ft
Massive	Very Wide	> 10 ft

Table 3A

SUMMARY OF LABORATORY RESULTS

Sample Information						Atterberg Limits			
Location	Sample	Depth, ft	Elevation, ft	Moisture Content, %	Dry Unit Weight, pcf	Liquid Limit, %	Plasticity Index, %	Fines Content, %	Soil Type
B-1	S-1	2.5	--	43	--	--	--	--	FILL
	S-2	5.0	--	35	--	--	--	--	SILT
	S-2	6.0	--	31	84	--	--	--	SILT
	S-3	7.0	--	30	--	--	--	--	SILT
	S-4	10.0	--	34	--	--	--	--	SILT
	S-5	12.5	--	48	--	--	--	--	Silty SAND
	S-6	15.0	--	58	--	56	13	46	Silty SAND
	S-7	20.0	--	54	--	--	--	--	Silty SAND
	S-8	25.0	--	57	--	--	--	44	Silty SAND
	S-9	30.0	--	51	--	--	--	--	Silty SAND
	S-10	35.0	--	54	--	--	--	--	SANDSTONE
	S-11	40.0	--	40	--	--	--	--	SANDSTONE
B-2	S-1	2.5	--	43	--	--	--	--	FILL
	S-2	5.5	--	32	--	--	--	83	SILT
	S-2	6.0	--	32	85	--	--	--	SILT
	S-3	7.0	--	35	--	--	--	--	SILT
	S-4	10.0	--	54	--	--	--	57	Sandy SILT
	S-5	12.5	--	57	--	--	--	--	Sandy SILT
	S-6	15.0	--	56	--	--	--	--	Sandy SILT
	S-7	20.0	--	57	--	--	--	51	Sandy SILT
	S-8	25.0	--	57	--	--	--	--	Sandy SILT
	S-9	30.0	--	50	--	--	--	--	SANDSTONE
	S-10	35.0	--	34	--	--	--	--	SANDSTONE
B-3	S-1	2.5	--	31	--	--	--	--	Sandy SILT
	S-2	5.0	--	41	--	--	--	68	Sandy SILT
	S-3	8.5	--	55	--	--	--	--	Silty SAND
	S-3	9.0	--	54	66	--	--	--	Silty SAND
	S-4	9.5	--	54	--	--	--	--	Silty SAND
	S-5	12.5	--	62	--	--	--	43	Silty SAND
	S-6	15.0	--	61	--	--	--	--	Silty SAND
	S-7	20.0	--	47	--	--	--	--	Silty SAND
	S-8	25.0	--	56	--	--	--	--	Silty SAND
	S-9	30.0	--	52	--	--	--	--	Silty SAND
	S-10	35.0	--	41	--	--	--	--	SANDSTONE
B-4	S-1	2.5	--	27	--	--	--	--	SILT
	S-2	5.0	--	39	--	--	--	--	SILT
	S-3	7.5	--	43	--	--	--	--	SILT
	S-3	9.0	--	37	82	--	--	--	SILT
	S-4	9.5	--	42	--	--	--	--	Silty SAND
	S-5	12.5	--	55	--	--	--	37	Silty SAND

Table 3A

SUMMARY OF LABORATORY RESULTS

Sample Information						Atterberg Limits			
Location	Sample	Depth, ft	Elevation, ft	Moisture Content, %	Dry Unit Weight, pcf	Liquid Limit, %	Plasticity Index, %	Fines Content, %	Soil Type
B-4	S-6	15.0	--	54	--	--	--	--	Silty SAND
	S-7	20.0	--	55	--	--	--	--	Silty SAND
	S-8	25.0	--	58	--	--	--	34	Silty SAND
	S-9	30.0	--	57	--	--	--	--	Silty SAND
	S-10	35.0	--	48	--	--	--	--	SANDSTONE
B-5	S-1	2.5	--	27	--	--	--	--	SILT
	S-2	5.0	--	39	--	35	2	--	SILT
	S-3	8.0	--	44	--	--	--	--	Silty SAND
	S-3	8.5	--	44	78	--	--	--	Silty SAND
	S-4	9.5	--	51	--	--	--	--	Silty SAND
	S-5	12.5	--	66	--	--	--	34	Silty SAND
	S-6	15.0	--	59	--	--	--	--	Silty SAND
	S-7	20.0	--	50	--	--	--	--	Silty SAND
	S-8	25.0	--	54	--	--	--	--	Silty SAND
	S-9	30.0	--	54	--	--	--	36	Silty SAND
	S-10	35.0	--	42	--	--	--	--	SANDSTONE
	S-11	40.0	--	42	--	--	--	--	SANDSTONE
B-6	S-1	2.5	--	29	--	--	--	--	SILT
	S-3	7.5	--	33	--	28	NP	--	SILT
	S-4	10.0	--	40	--	--	--	--	Sandy SILT
	S-5	12.5	--	52	--	--	--	--	Silty SAND
	S-6	15.0	--	50	--	--	--	33	Silty SAND
	S-7	20.0	--	45	--	--	--	--	Silty SAND
	S-8	25.0	--	53	--	--	--	--	Silty SAND
	S-9	30.0	--	53	--	--	--	--	SANDSTONE
B-7	S-1	5.0	--	23	--	--	--	--	SILT
	S-2	10.0	--	41	--	--	--	--	Sandy SILT
	S-3	15.0	--	32	--	--	--	--	Sandy SILT
B-8	S-1	12.5	--	56	--	--	--	53	Sandy SILT
B-9	S-1	14.5	--	37	--	--	--	62	Sandy SILT
HA-1	S-1	2.0	--	35	--	--	--	--	FILL
	S-2	4.0	--	27	--	--	--	--	SILT
HA-2	S-1	2.0	--	32	--	--	--	--	FILL
	S-2	4.0	--	24	--	--	--	--	SILT

GRI

BORING AND TEST PIT LOG LEGEND

SOIL SYMBOLS

Symbol	Typical Description
	LANDSCAPE MATERIALS
	FILL
	GRAVEL; clean to some silt, clay, and sand
	Sandy GRAVEL; clean to some silt and clay
	Silty GRAVEL; up to some clay and sand
	Clayey GRAVEL; up to some silt and sand
	SAND; clean to some silt, clay, and gravel
	Gravelly SAND; clean to some silt and clay
	Silty SAND; up to some clay and gravel
	Clayey SAND; up to some silt and gravel
	SILT; up to some clay, sand, and gravel
	Gravelly SILT; up to some clay and sand
	Sandy SILT; up to some clay and gravel
	Clayey SILT; up to some sand and gravel
	CLAY; up to some silt, sand, and gravel
	Gravelly CLAY; up to some silt and sand
	Sandy CLAY; up to some silt and gravel
	Silty CLAY; up to some sand and gravel
	PEAT

BEDROCK SYMBOLS

Symbol	Typical Description
	BASALT
	MUDSTONE
	SILTSTONE
	SANDSTONE

SURFACE MATERIAL SYMBOLS

Symbol	Typical Description
	Asphalt concrete PAVEMENT
	Portland cement concrete PAVEMENT
	Crushed rock BASE COURSE

SAMPLER SYMBOLS

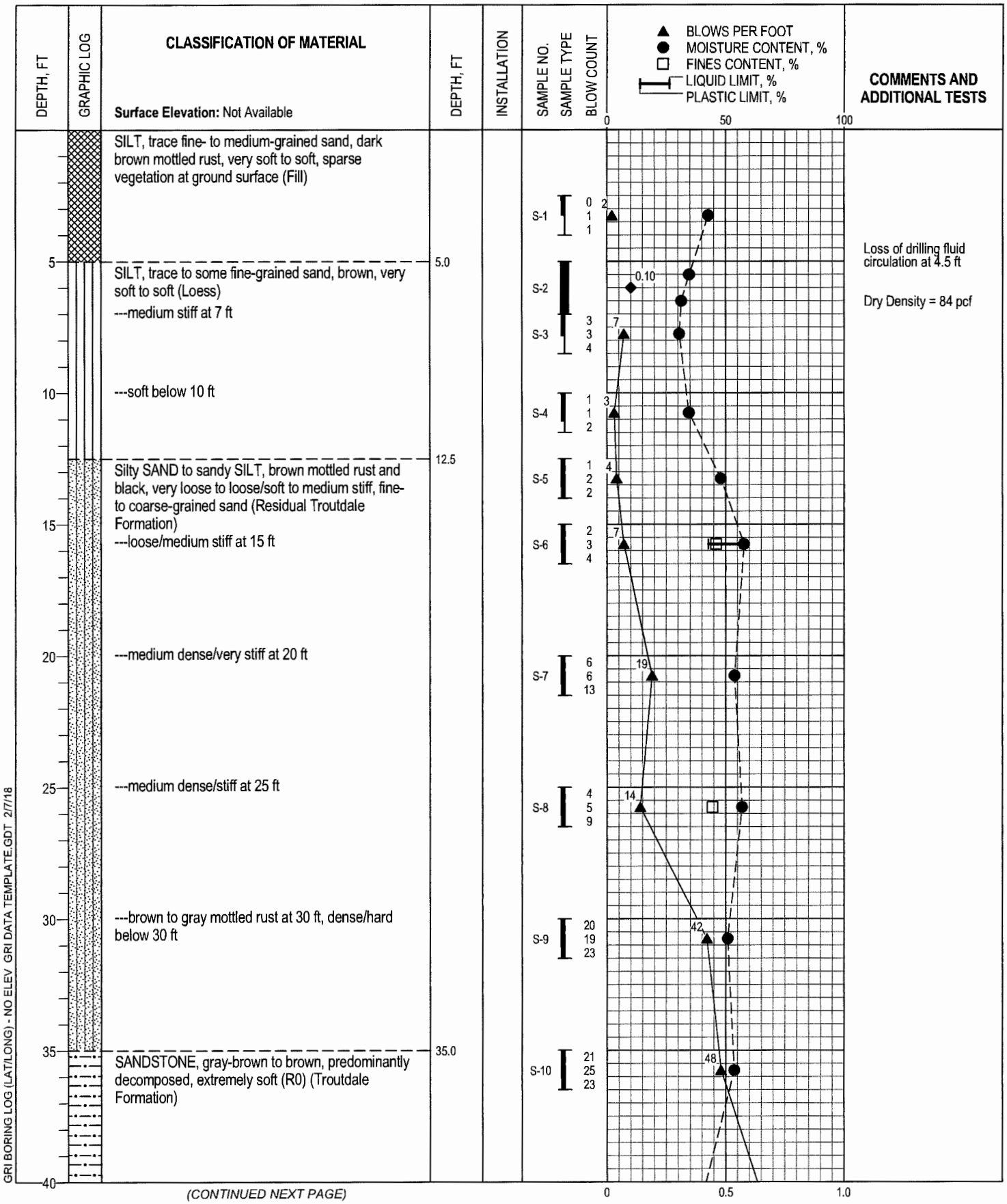
Symbol	Sampler Description
	2.0-in. O.D. split-spoon sampler and Standard Penetration Test with recovery (ASTM D1586)
	Shelby tube sampler with recovery (ASTM D1587)
	3.0-in. O.D. split-spoon sampler with recovery (ASTM D3550)
	Grab Sample
	Rock core sample interval
	Sonic core sample interval
	Geoprobe sample interval

INSTALLATION SYMBOLS

Symbol	Symbol Description
	Flush-mount monument set in concrete
	Concrete, well casing shown where applicable
	Bentonite seal, well casing shown where applicable
	Filter pack, machine-slotted well casing shown where applicable
	Grout, vibrating-wire transducer cable shown where applicable
P	Vibrating-wire pressure transducer
	1-in.-diameter solid PVC
	1-in.-diameter hand-slotted PVC
	Grout, inclinometer casing shown where applicable

FIELD MEASUREMENTS

Symbol	Typical Description
▽	Groundwater level during drilling and date measured
▼	Groundwater level after drilling and date measured
	Rock core recovery (%)
	Rock quality designation (RQD, %)

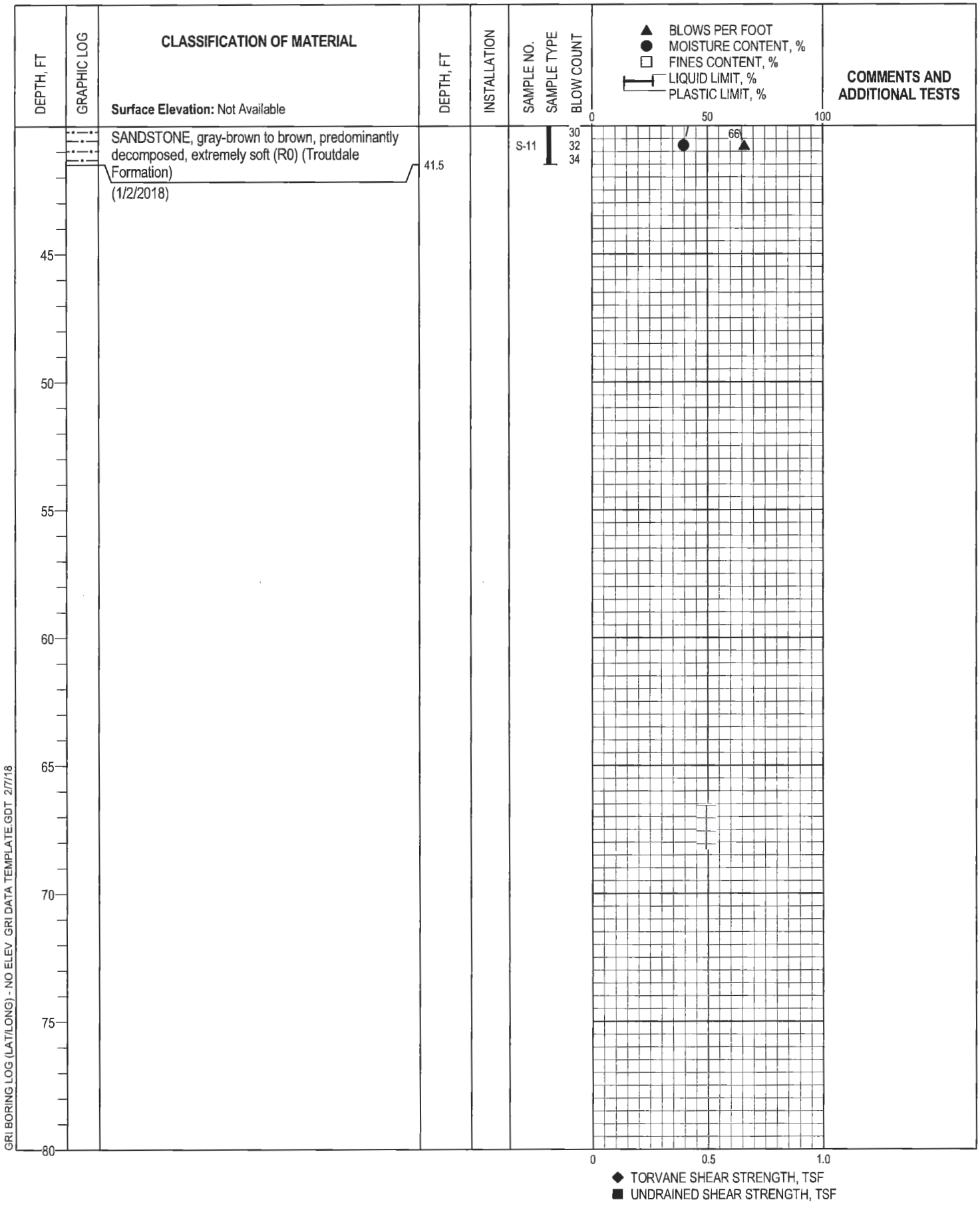


GRI BORING LOG (LAT/LONG) - NO ELEV GRI DATA TEMPLATE.GDT 2/7/18

Logged By: A. Baumann	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 1/2/18	**Coordinates:** Not Available
Drilling Method: Mud Rotary **Equipment:** CME 75 HT Truck-Mounted Drill Rig **Hole Diameter:** 5 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 0.8
Note: See Legend for Explanation of Symbols	

BORING B-1

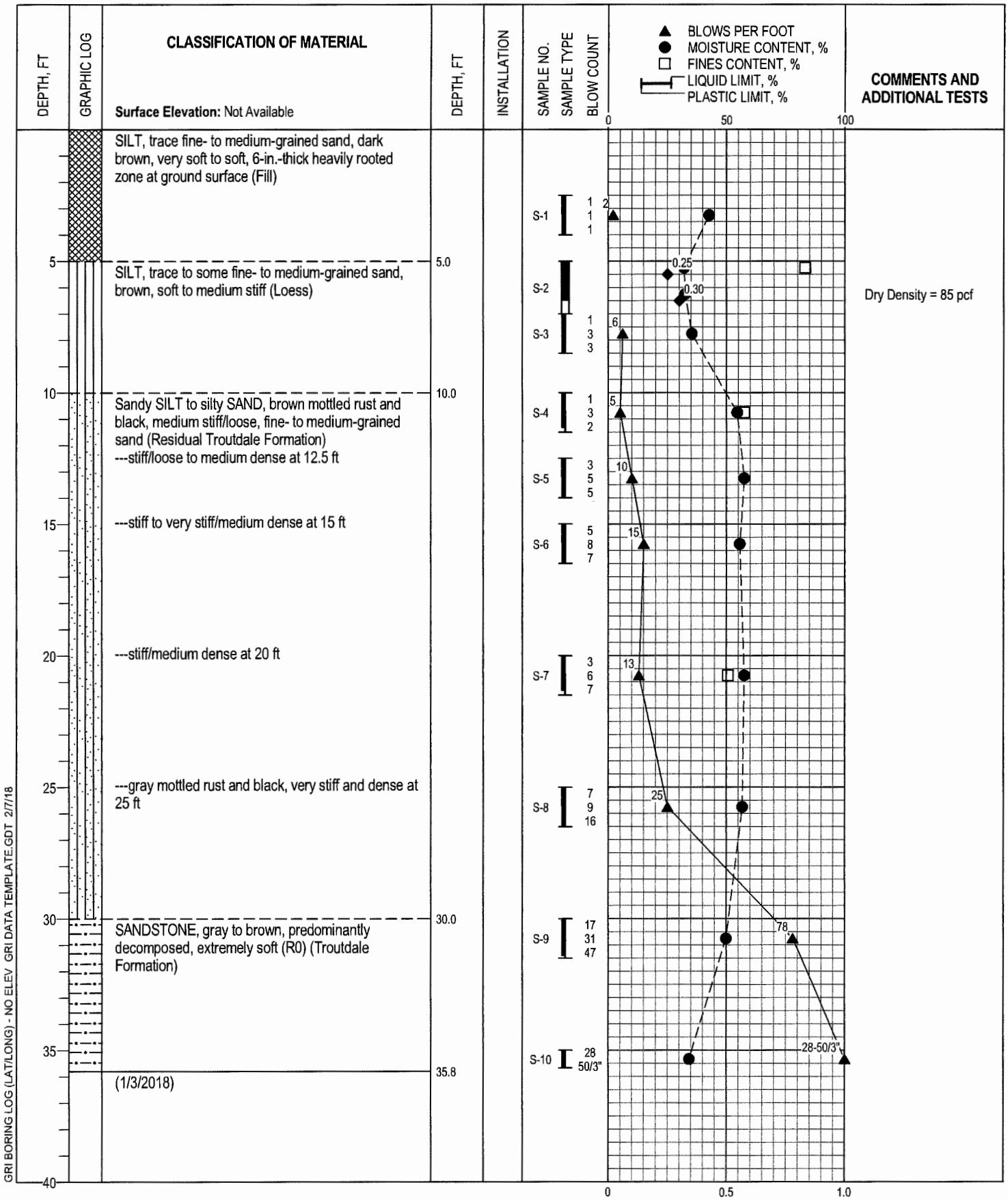
DEC. 2018 JOB NO. 6060 FIG. 1A



GRI

BORING B-1

DEC. 2018 JOB NO. 6060 FIG. 1A

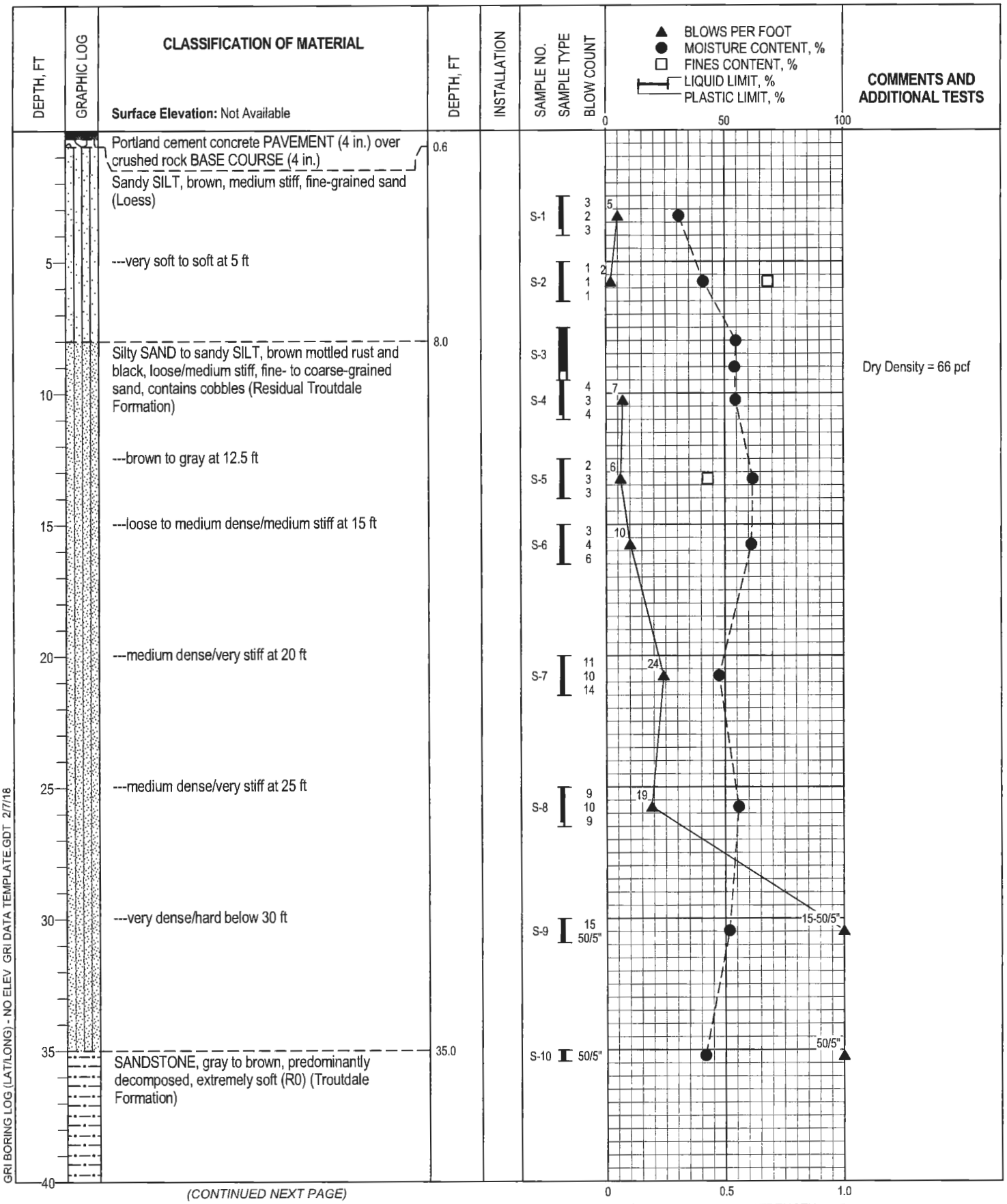


GRI BORING LOG (LAT/LONG) - NO ELEV GRI DATA TEMPLATE.GDT 2/7/18

Logged By: T. Bineham	**Drilled by:** Western States Soil Conservation, Inc.	
Date Started: 1/3/18	**Coordinates:** Not Available	
Drilling Method: Mud Rotary **Equipment:** CME 75 HT Truck-Mounted Drill Rig **Hole Diameter:** 5 in.		**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 0.8
Note: See Legend for Explanation of Symbols		

BORING B-2

DEC. 2018 JOB NO. 6060 FIG. 2A

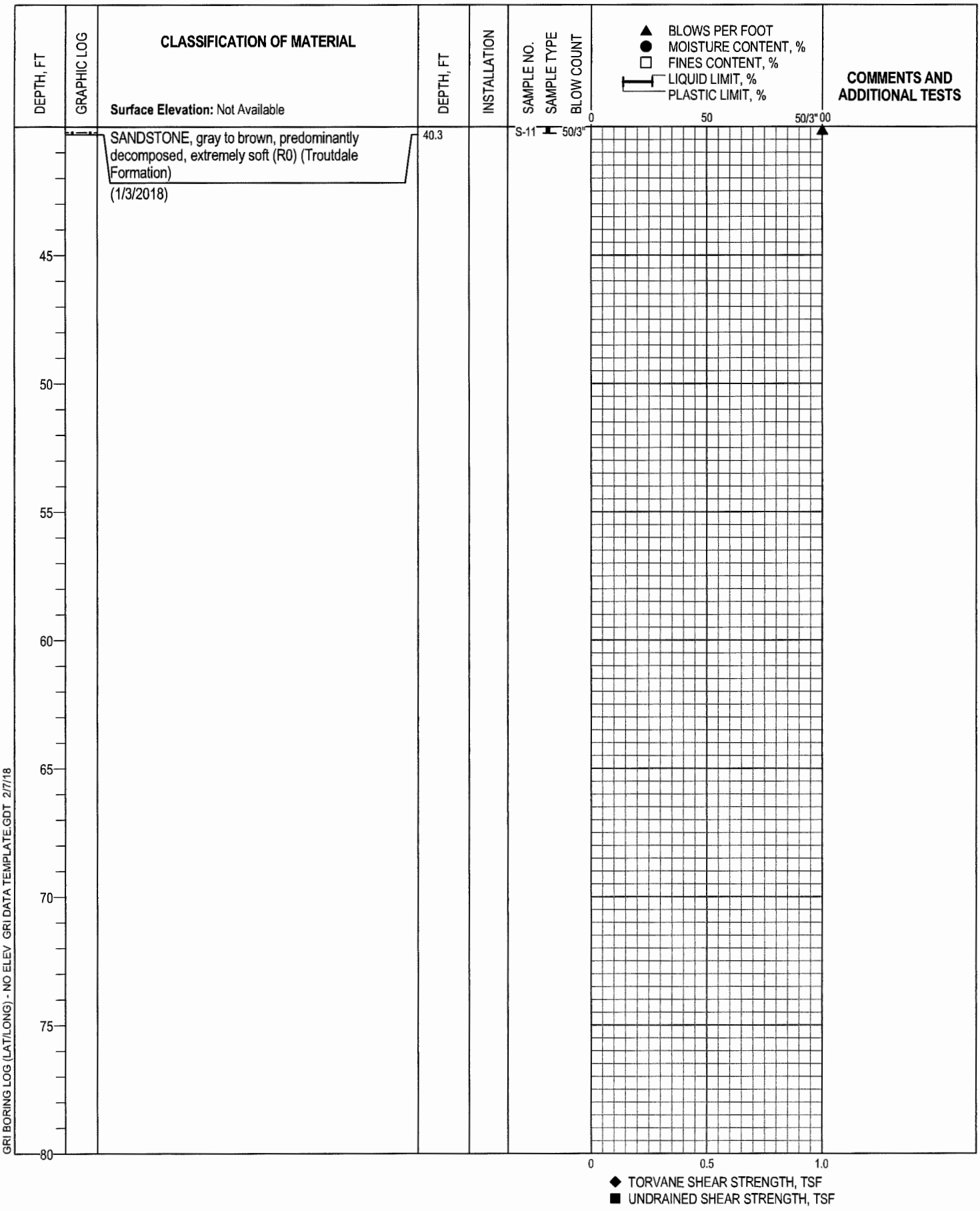


(CONTINUED NEXT PAGE)

Logged By: T. Bineham	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 1/3/18	**Coordinates:** Not Available
Drilling Method: Mud Rotary **Equipment:** CME 75 HT Truck-Mounted Drill Rig **Hole Diameter:** 5 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 0.8
Note: See Legend for Explanation of Symbols	

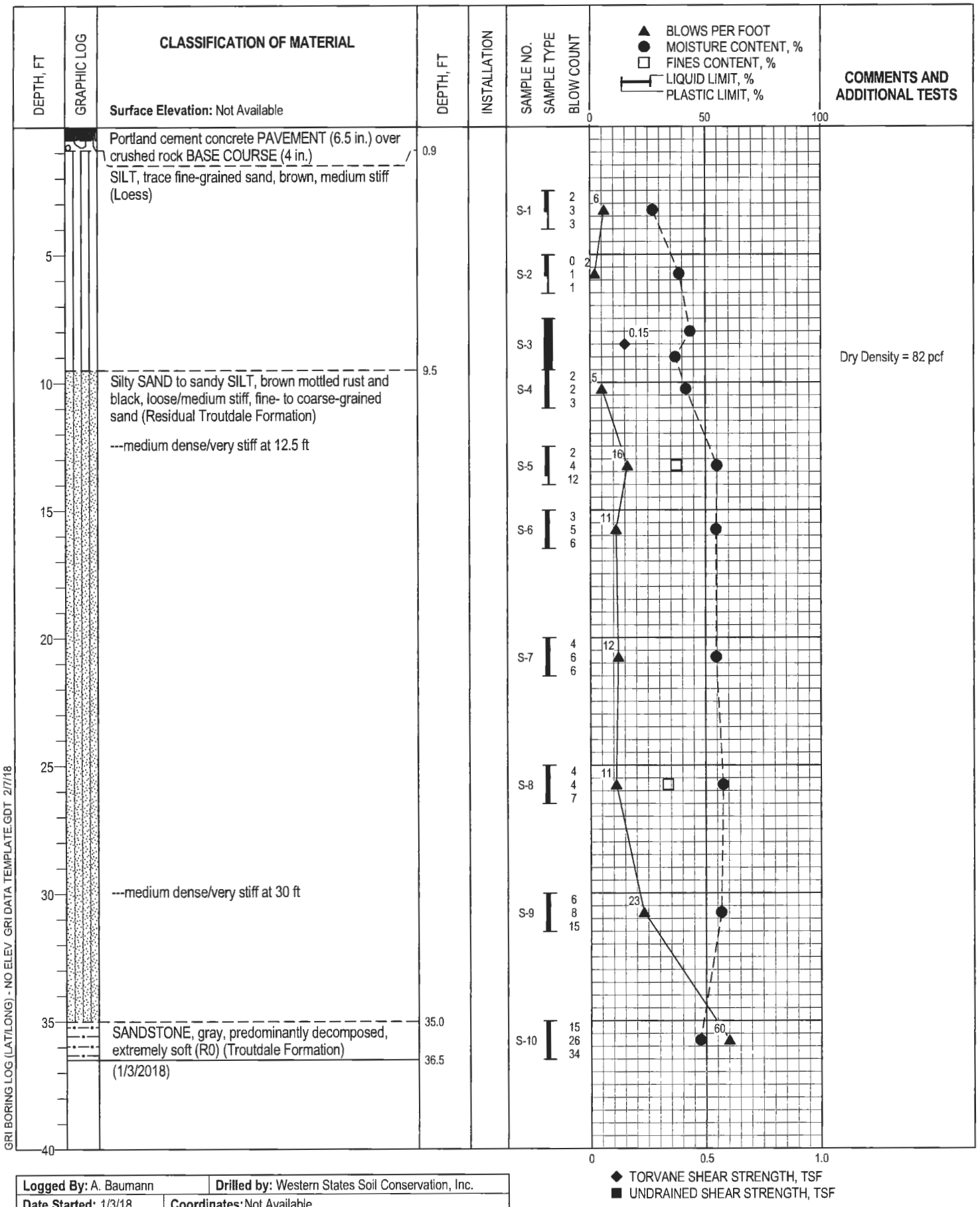
BORING B-3

DEC. 2018 JOB NO. 6060 FIG. 3A



BORING B-3

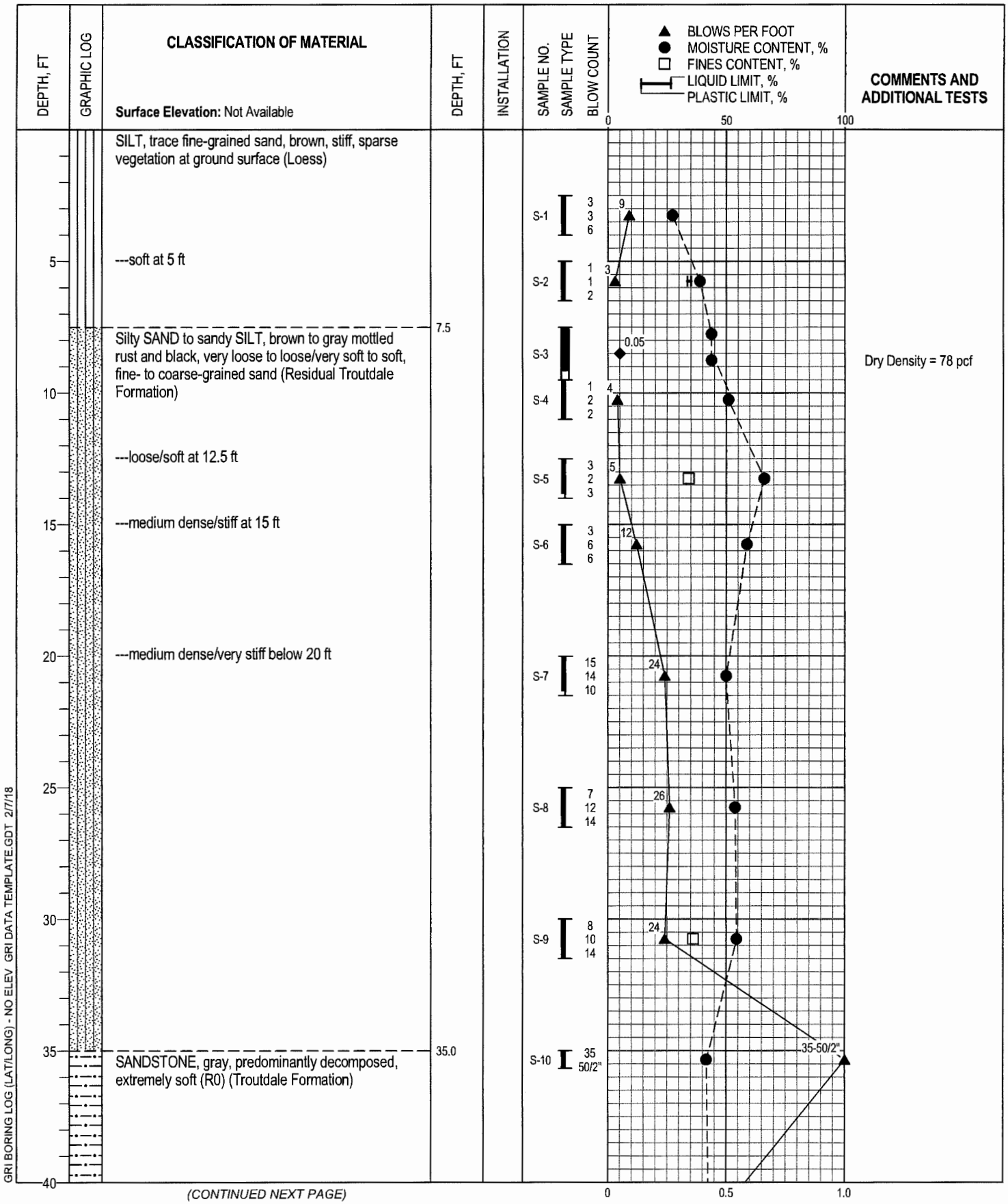
DEC. 2018 JOB NO. 6060 FIG. 3A



Logged By: A. Baumann	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 1/3/18	**Coordinates:** Not Available
Drilling Method: Mud Rotary **Equipment:** CME 55 HT Truck-Mounted Drill Rig **Hole Diameter:** 5 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 0.8
Note: See Legend for Explanation of Symbols	

BORING B-4

DEC. 2018 JOB NO. 6060 FIG. 4A



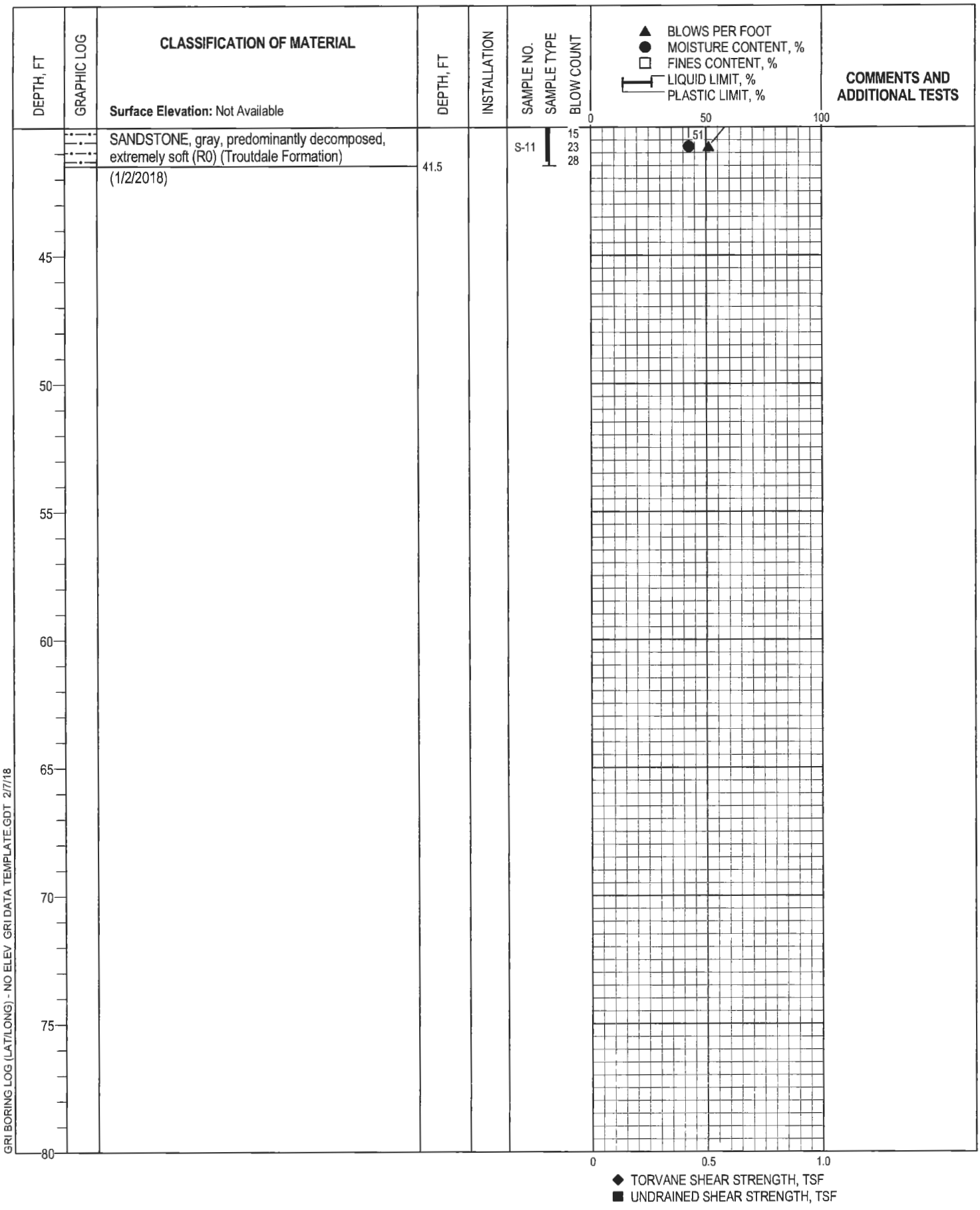
(CONTINUED NEXT PAGE)

GRI BORING LOG (LAT/LONG) - NO ELEV GRI DATA TEMPLATE.GDT 2/7/18

Logged By: T. Bineham	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 1/2/18	**Coordinates:** Not Available
Drilling Method: Mud Rotary **Equipment:** CME 55 HT Truck-Mounted Drill Rig **Hole Diameter:** 5 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 0.8
Note: See Legend for Explanation of Symbols	

BORING B-5

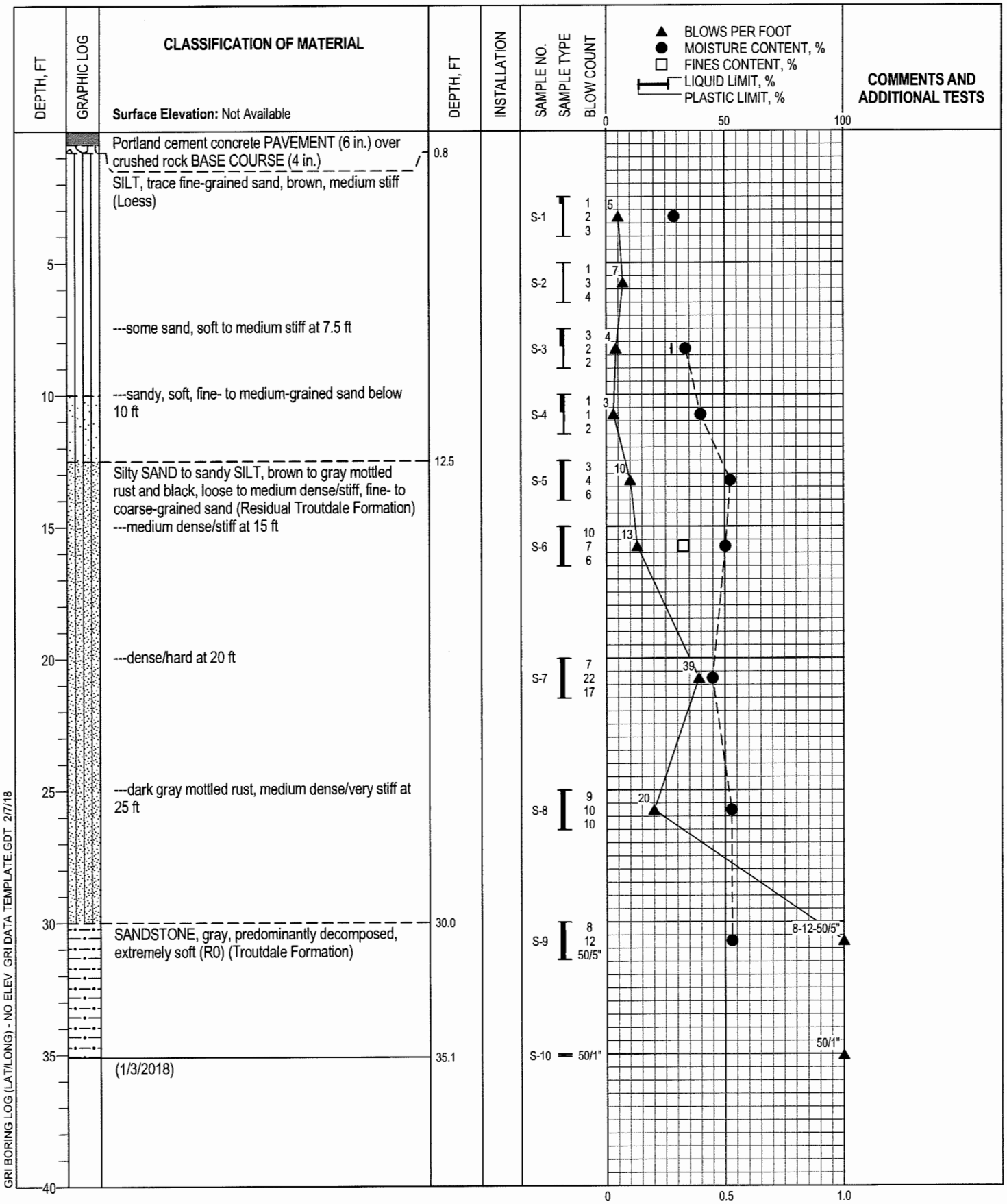
DEC. 2018 JOB NO. 6060 FIG. 5A



GRI

BORING B-5

DEC. 2018 JOB NO. 6060 FIG. 5A

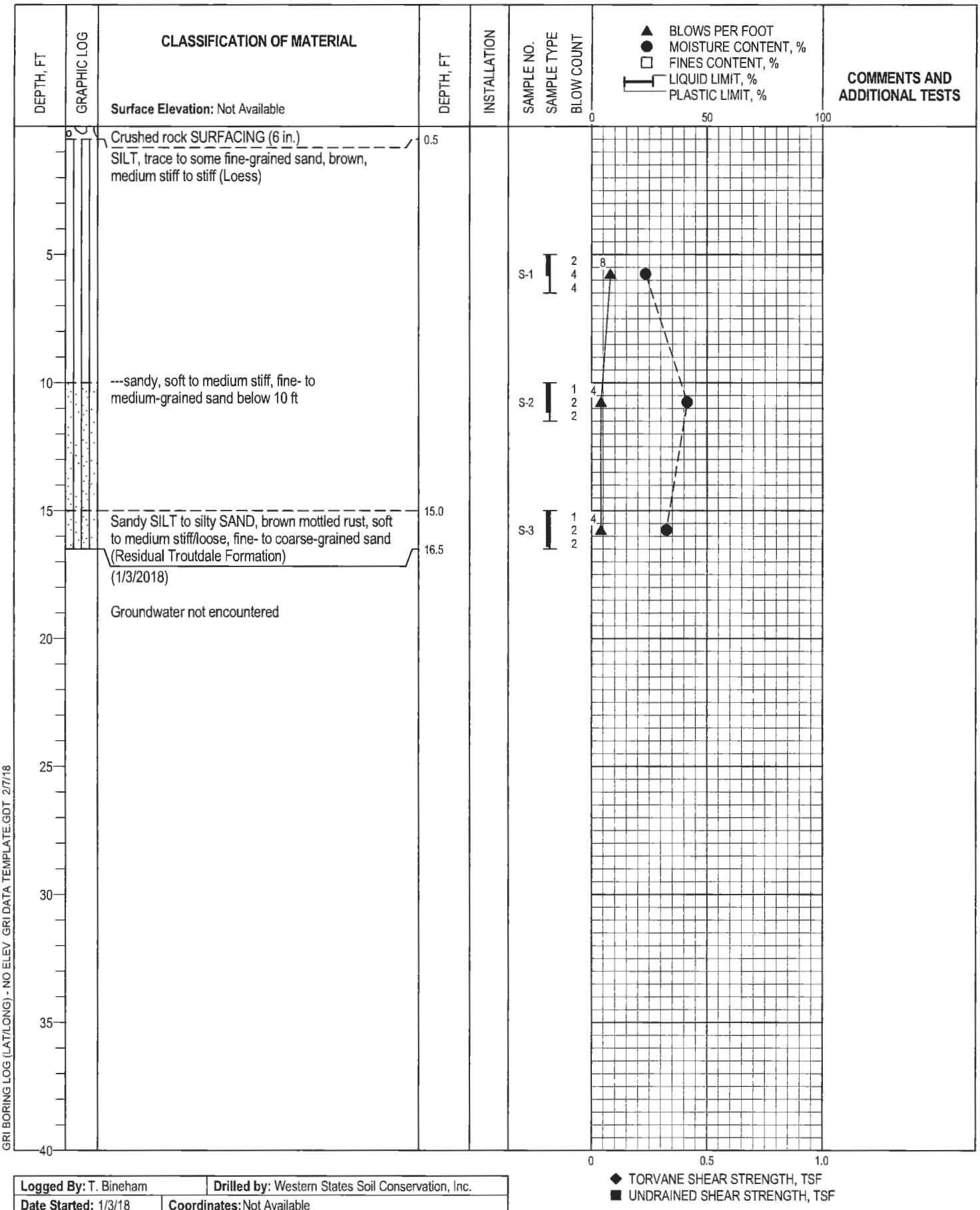


GRI BORING LOG (LAT/LONG) - NO ELEV GRI DATA TEMPLATE.GDT 2/7/18

Logged By: A. Baumann	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 1/3/18	**Coordinates:** Not Available
Drilling Method: Mud Rotary **Equipment:** CME 55 HT Truck-Mounted Drill Rig **Hole Diameter:** 5 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 0.8
Note: See Legend for Explanation of Symbols	

BORING B-6

DEC. 2018 JOB NO. 6060 FIG. 6A

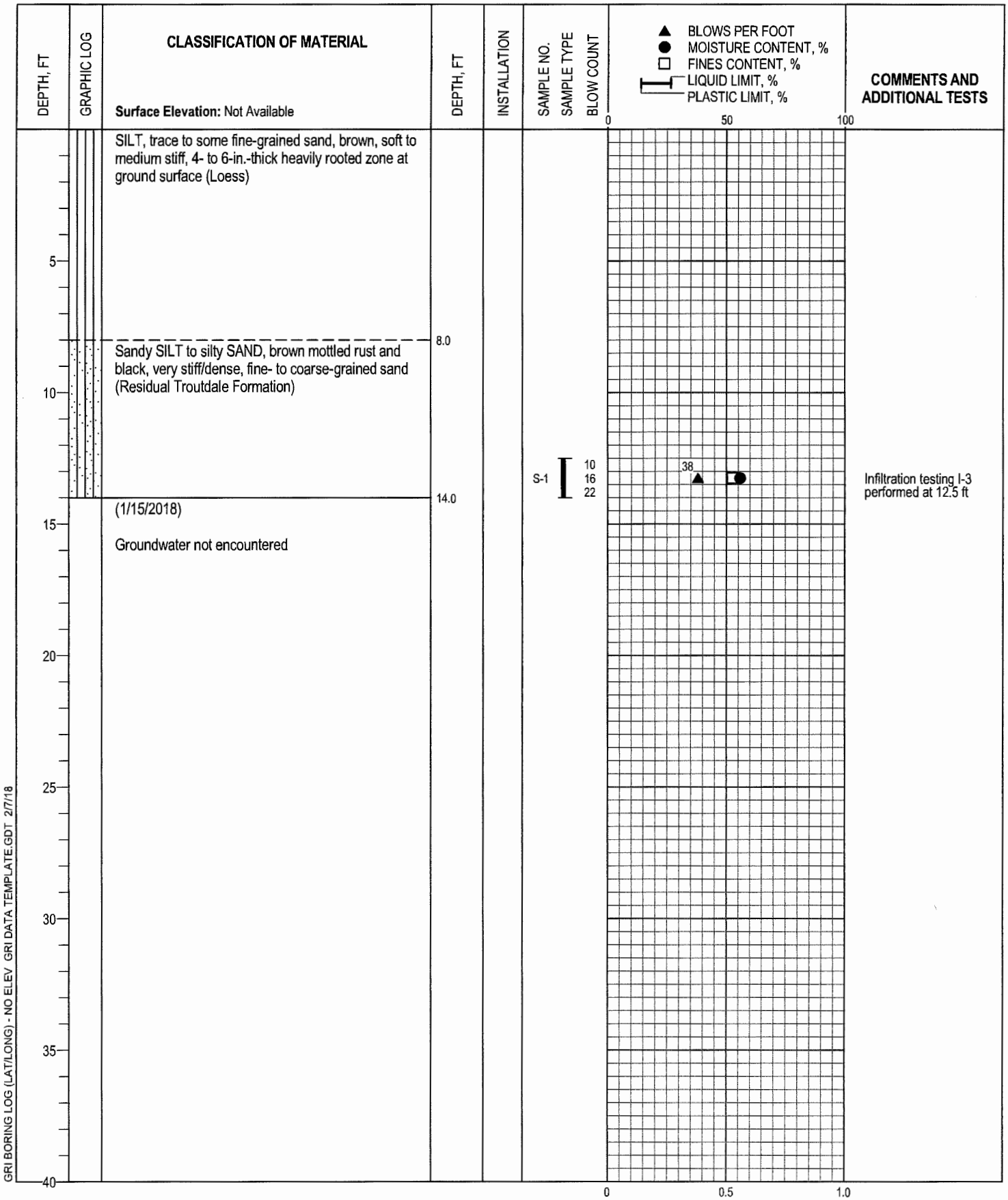


GRI BORING LOG (LAT/LONG) - NO ELEV GRI DATA TEMPLATE.GDT 2/7/18

Logged By: T. Bineham	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 1/3/18	**Coordinates:** Not Available
Drilling Method: Hollow-Stem Auger **Equipment:** CME 75 HT Truck-Mounted Drill Rig **Hole Diameter:** 4 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 0.8
Note: See Legend for Explanation of Symbols	

BORING B-7

DEC. 2018 JOB NO. 6060 FIG. 7A



GRI BORING LOG (LAT/LONG) - NO ELEV GRI DATA TEMPLATE.GDT 2/7/18

Logged By: A. Baumann	**Drilled by:** Greg Vandehey Soil Sampling
Date Started: 1/15/18	**Coordinates:** Not Available
Drilling Method: Hollow-Stem Auger **Equipment:** Simco 2800SK Trailer-Mounted Drill Rig **Hole Diameter:** 6 in.	**Hammer Type:** Cat Head **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:**
Note: See Legend for Explanation of Symbols	

BORING B-8

DEC. 2018 JOB NO. 6060 FIG. 8A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	DEPTH, FT	INSTALLATION	SAMPLE NO. / SAMPLE TYPE / BLOW COUNT	▲ BLOWS PER FOOT ● MOISTURE CONTENT, % □ FINES CONTENT, % LIQUID LIMIT, % PLASTIC LIMIT, % 0 50 100	COMMENTS AND ADDITIONAL TESTS
		Surface Elevation: Not Available					
0		SILT, trace to some fine- to coarse-grained sand, brown, soft to medium stiff (Loess)					
5							
10							
15		---sandy, medium stiff to stiff, fine- to medium-grained sand below 14.5 ft			S-1 4 4 4	8 ▲ ● □	Infiltration testing I-4 performed at 14.5 ft
		(1/15/2018) Groundwater not encountered	16.0				
20							
25							
30							
35							
40						0 0.5 1.0 ◆ TORVANE SHEAR STRENGTH, TSF ■ UNDRAINED SHEAR STRENGTH, TSF	

GRI BORING LOG (LAT/LONG) - NO ELEV GRI DATA TEMPLATE.GDT 2/7/18

Logged By: A. Baumann	**Drilled by:** Greg Vandehey Soil Sampling
Date Started: 1/15/18	**Coordinates:** Not Available
Drilling Method: Hollow-Stem Auger **Equipment:** Simco 2800SK Trailer-Mounted Drill Rig **Hole Diameter:** 6 in.	**Hammer Type:** Cat Head **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:**
Note: See Legend for Explanation of Symbols	

GRI

BORING B-9

DEC. 2018 JOB NO. 6060 FIG. 9A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	DEPTH, FT	SAMPLE NO. SAMPLE TYPE	MOISTURE CONTENT, % / FINES CONTENT, % / LIQUID LIMIT, % / PLASTIC LIMIT, %	COMMENTS AND ADDITIONAL TESTS
					0 50 100	

HA-1 **Surface Elevation:** Not Available

0		SILT, trace fine to coarse-grained sand, dark brown, medium stiff, contains organics (Fill)		
		SILT, trace fine-grained sand, brown, medium stiff (Loess)	2.5	S-1, S-2
5		(1/2/2018) Groundwater not encountered	5.0	Infiltration testing I-1 performed at 5 ft
10				

0 0.5 1.0
◆ TORVANE SHEAR STRENGTH, TSF

Logged By: T. Bineham | **Excavated by:** GRI | **Equipment:** Hand Auger
Date Started: 1/2/18 | **Coordinates:** Not Available | **Note:** See Legend for Explanation of Symbols

HA-2 **Surface Elevation:** Not Available

0		SILT, trace fine-grained sand, dark brown, medium stiff (Fill)		
		SILT, trace fine-grained sand, brown, medium stiff (Loess)	2.5	S-1, S-2
5		(1/2/2018) Groundwater not encountered	5.0	Infiltration testing I-2 performed at 5 ft
10				

0 0.5 1.0
◆ TORVANE SHEAR STRENGTH, TSF

Logged By: T. Bineham | **Excavated by:** GRI | **Equipment:** Hand Auger
Date Started: 1/2/18 | **Coordinates:** Not Available | **Note:** See Legend for Explanation of Symbols

GRI HA LOG (LAT/LONG) - 2 PP - NO ELEV GRI DATA TEMPLATE.GDT 2/6/18

GRI BORINGS

DEC. 2018 JOB NO. 6060 FIG. 10A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	DEPTH, FT	SAMPLE NO. SAMPLE TYPE	● MOISTURE CONTENT, % □ FINES CONTENT, % ⊢ LIQUID LIMIT, % ⊣ PLASTIC LIMIT, % 0 50 100	COMMENTS AND ADDITIONAL TESTS
HA-3		**Surface Elevation:** Not Available			0 50 100	
		SILT, trace fine- to medium-grained sand, brown, medium stiff (Loess)				
5						
		Sandy SILT to silty SAND, brown mottled rust and yellow, medium stiff/loose to medium dense, fine- to coarse-grained sand (Residual Troutdale Formation)	8.0			
10		(1/12/2018)	10.0			
		Groundwater not encountered				
					0 0.5 1.0 ◆ TORVANE SHEAR STRENGTH, TSF	

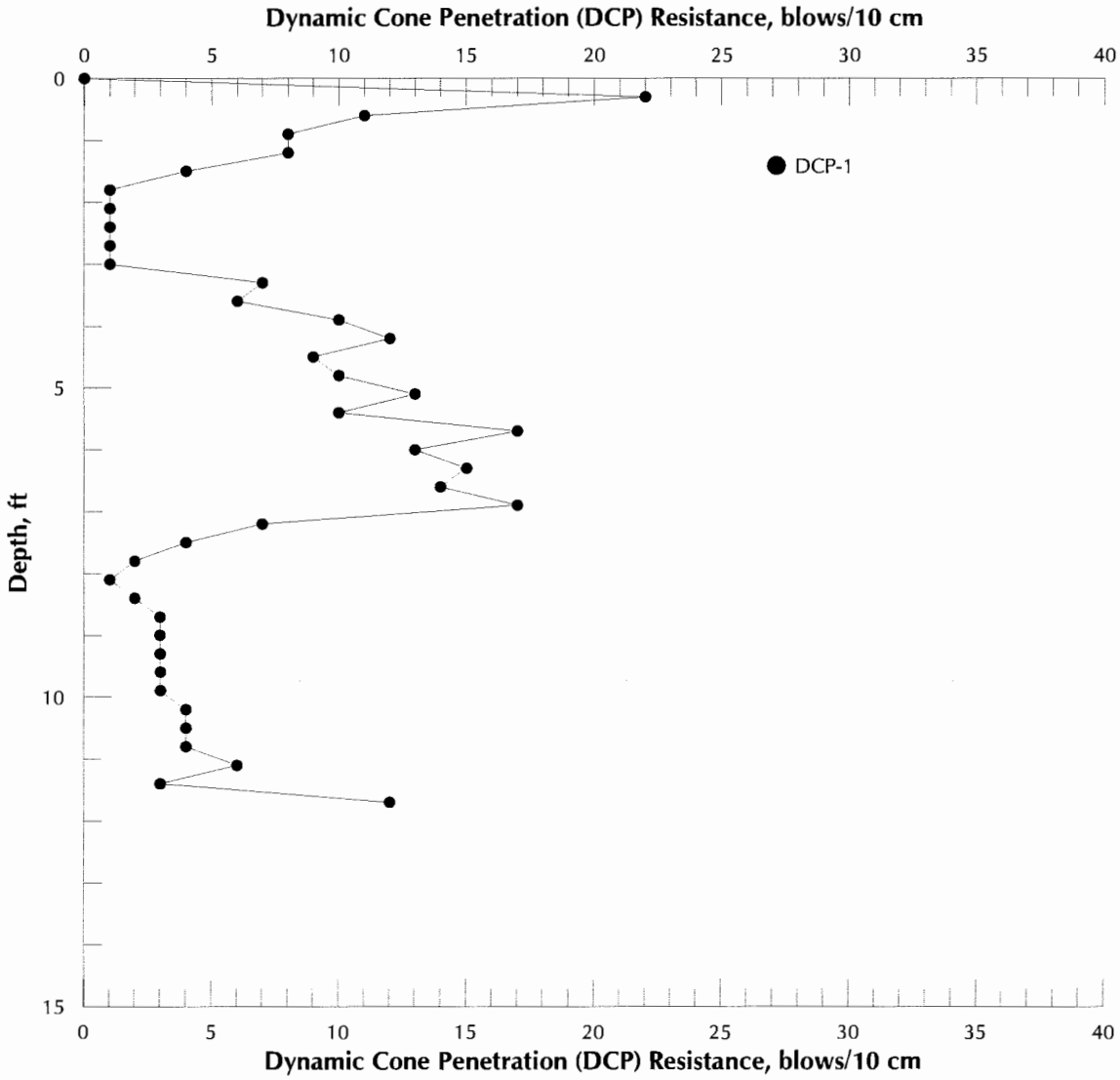
Logged By: T. Bineham | **Excavated by:** GRI | **Equipment:** Hand Auger

Date Started: 1/12/18 | **Coordinates:** Not Available | **Note:** See Legend for Explanation of Symbols

GRI HA LOG (LAT/LONG) - 2 PP - NO ELEV GRI DATA TEMPLATE.GDT 2/6/18

GRI BORINGS

DEC. 2018 JOB NO. 6060 FIG. 11A



GRI

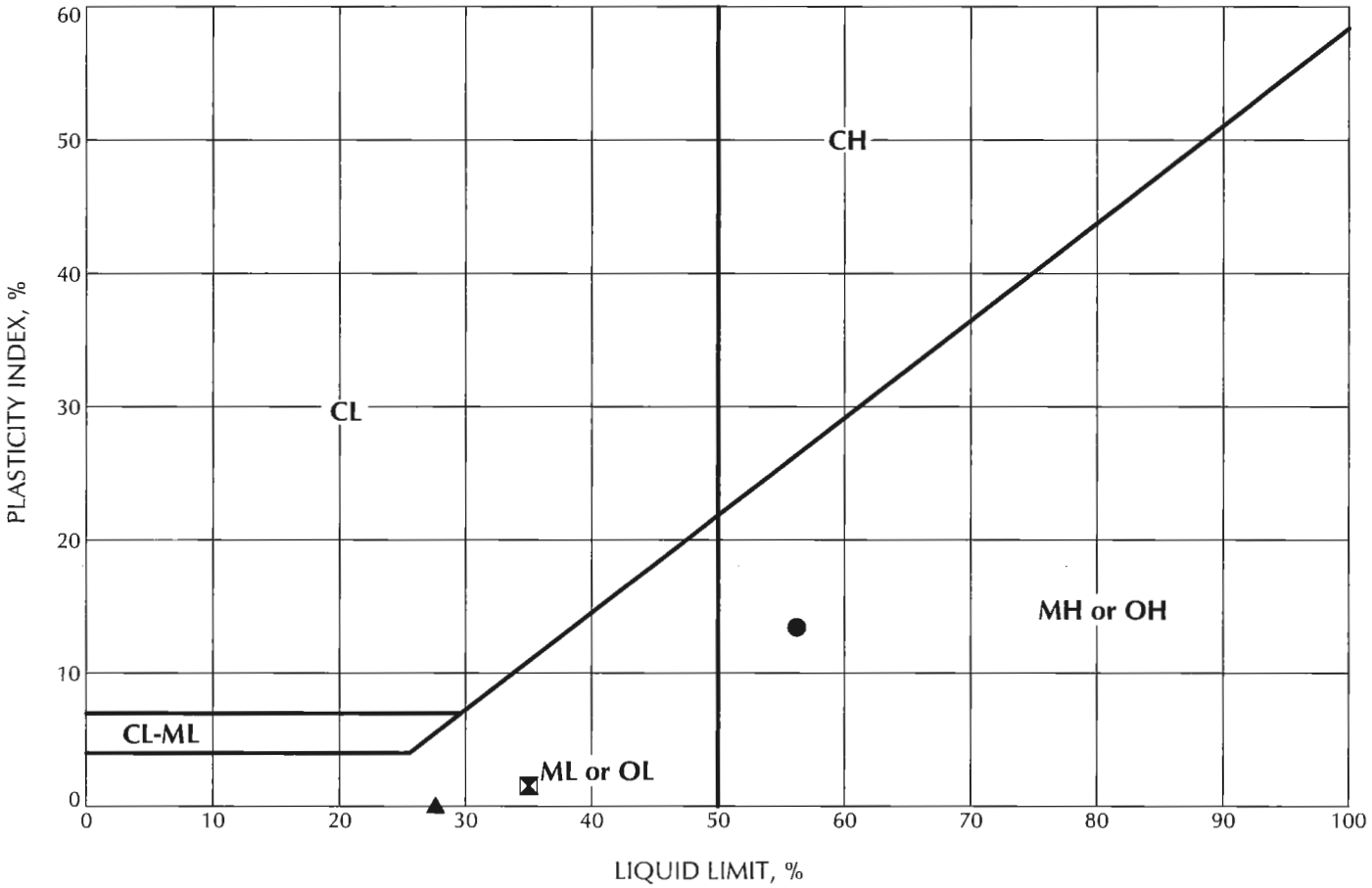
HSF, LLC
VIEW POINT INN

DYNAMIC CONE PENETRATION

DEC. 2018 JOB NO. 6060 FIG. 12A

GROUP SYMBOL	UNIFIED SOIL CLASSIFICATION FINE-GRAINED SOIL GROUPS
OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
ML	INORGANIC CLAYEY SILTS TO VERY FINE SANDS OF SLIGHT PLASTICITY
CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY

GROUP SYMBOL	UNIFIED SOIL CLASSIFICATION FINE-GRAINED SOIL GROUPS
OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
MH	INORGANIC SILTS AND CLAYEY SILT
CH	INORGANIC CLAYS OF HIGH PLASTICITY



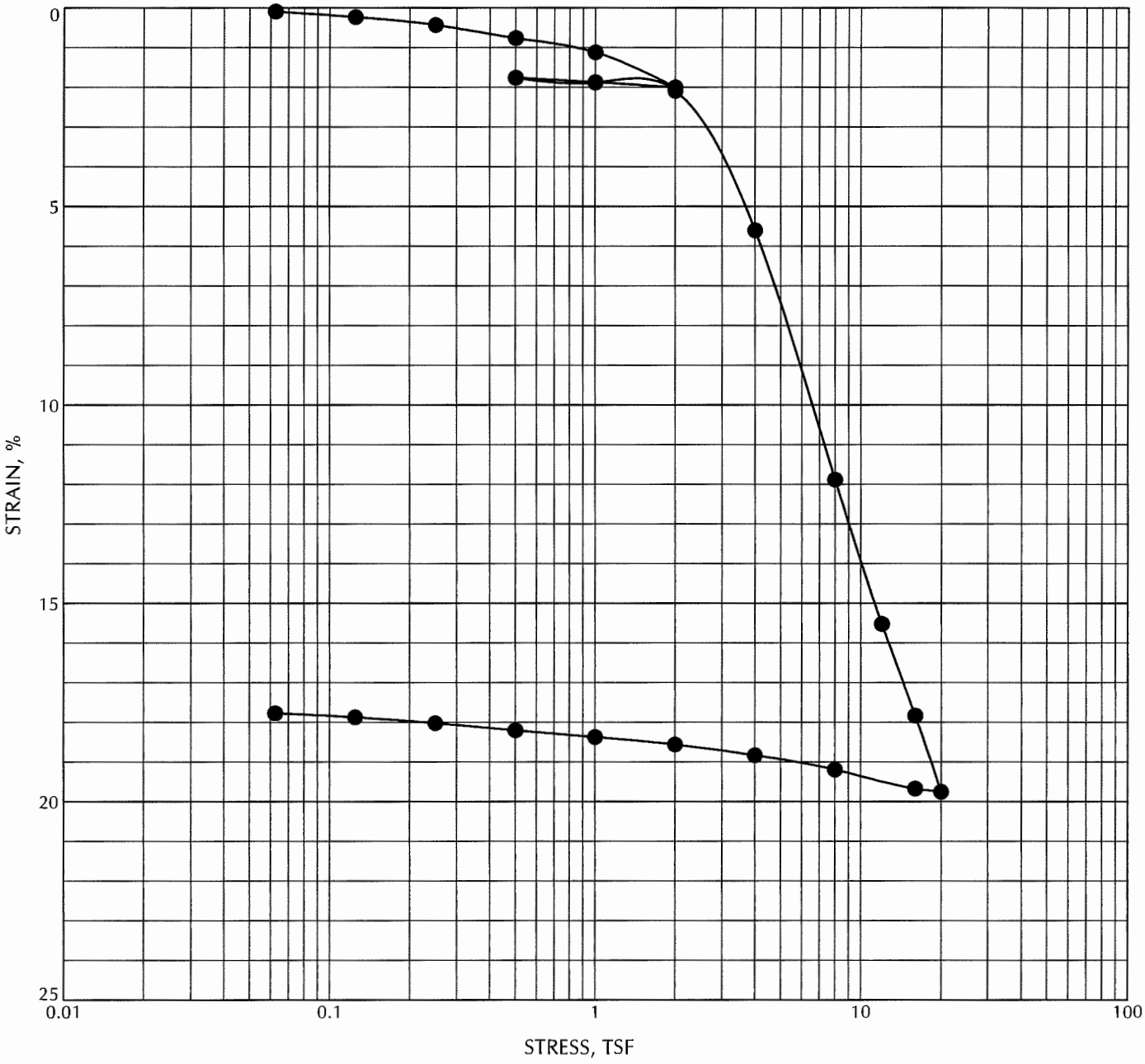
	Location	Sample	Depth, ft	Classification	LL	PL	PI	MC, %
●	B-1	S-6	15.0	Silty SAND, brown mottled rust and black, testing performed on portion finer than sieve #40 (Residual Troutdale Formation)	56	43	13	58
⊠	B-5	S-2	5.0	SILT, trace fine-grained sand, brown (Loess)	35	33	2	39
▲	B-6	S-3	7.5	SILT, some fine-grained sand, brown (Loess)	28	28	NP	33

GRI

PLASTICITY CHART

DEC. 2018 JOB NO. 6060 FIG. 13A

ATTERBERG-PLASTICITY 3 PER PAGE GRI DATA TEMPLATE.GDT 2/6/18



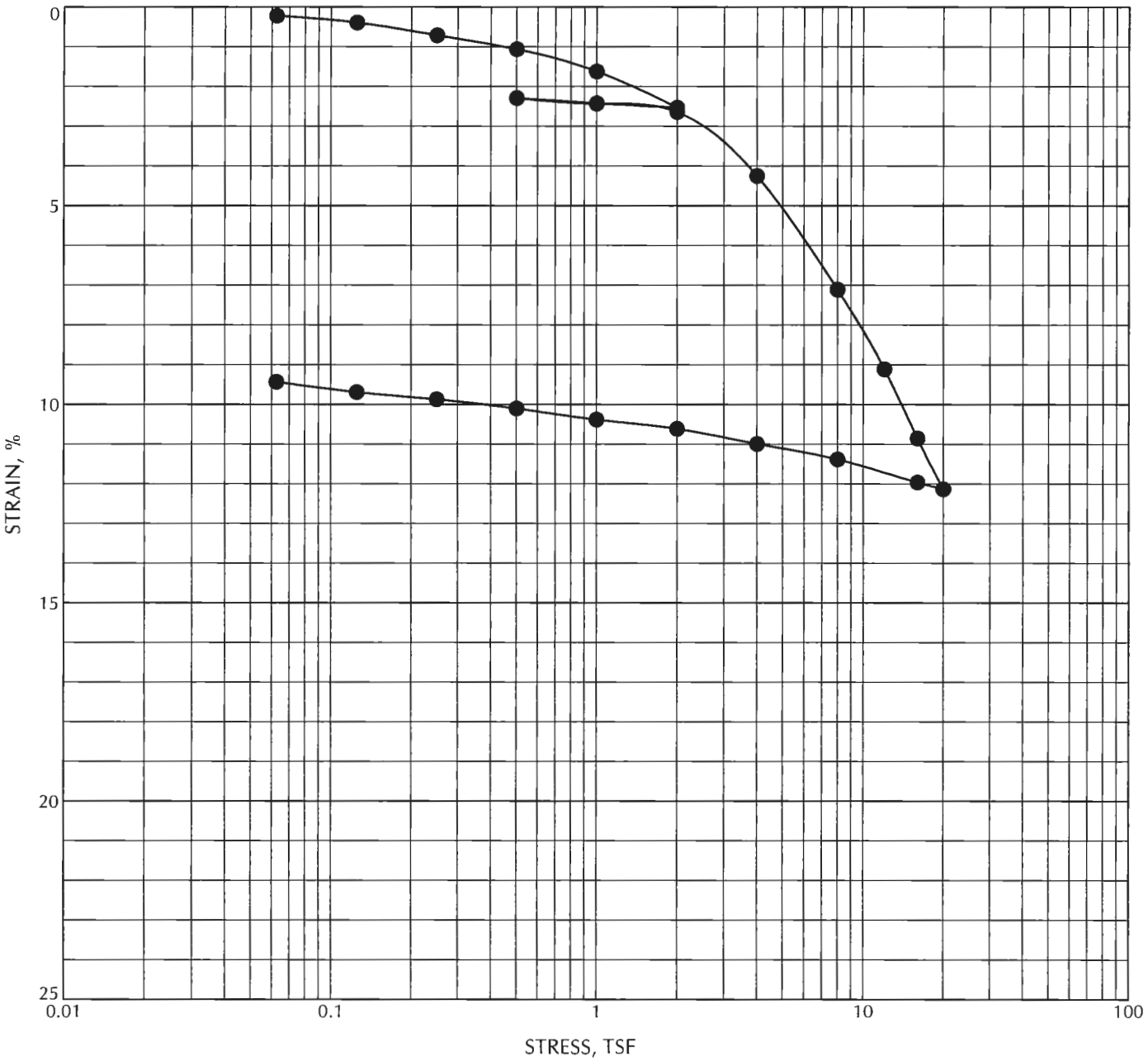
	Location	Sample	Depth, ft	Classification	Initial γ_d, pcf	Initial MC, %
●	B-1	S-2	5.5	SILT, trace fine-grained sand, brown, very soft to soft (Loess)	75	39

GRI

CONSOLIDATION TEST

DEC. 2018 JOB NO. 6060 FIG. 14A

CONSOL STRAIN GRI-0 TO 25-1 PER PAGE GRI DATA TEMPLATE.GDT 2/6/18



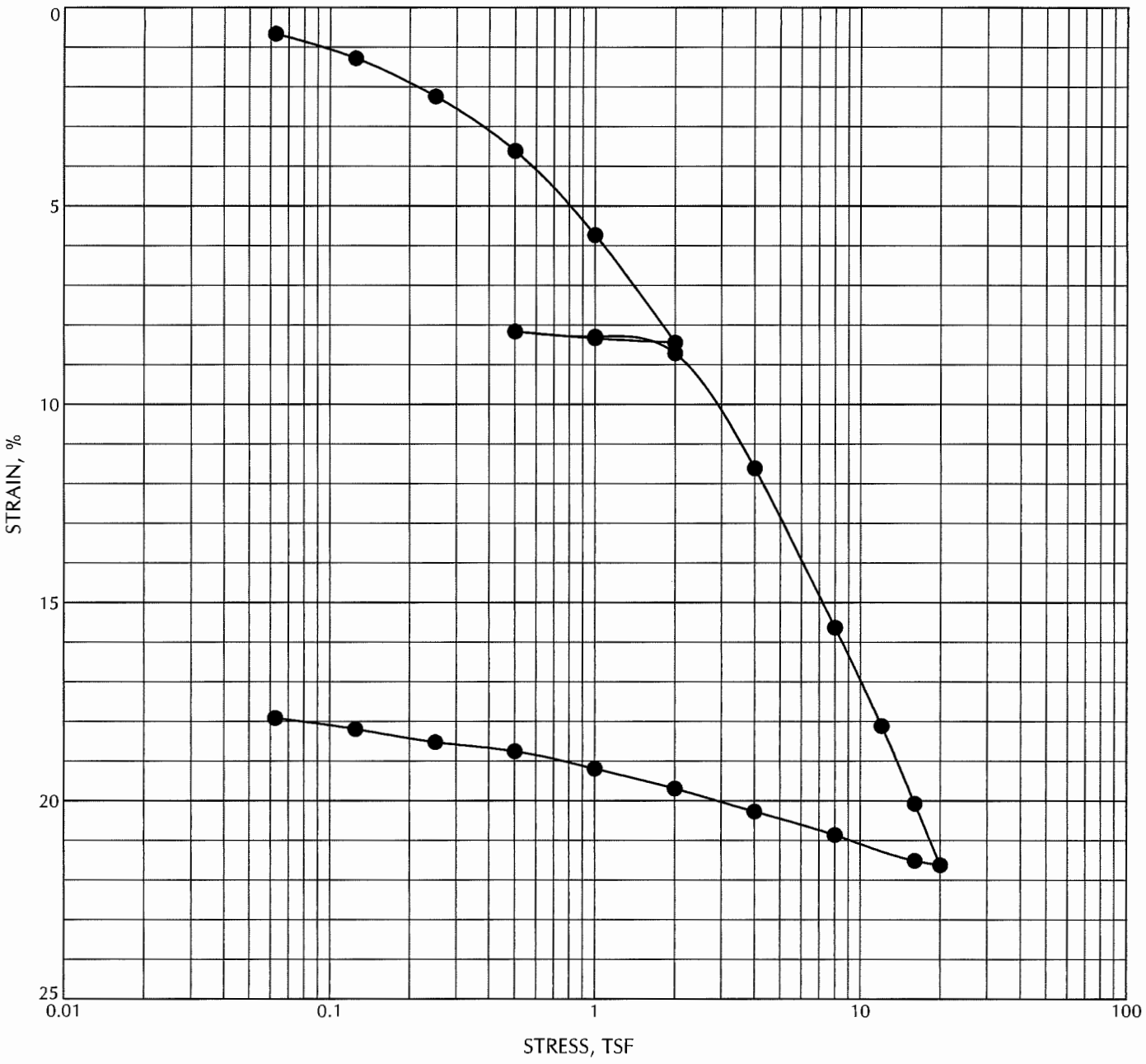
	Location	Sample	Depth, ft	Classification	Initial γ_d, pcf	Initial MC, %
●	B-2	S-2	6.25	SILT, some fine-grained sand, brown, soft (Loess)	85	33

GRI

CONSOLIDATION TEST

DEC. 2018 JOB NO. 6060 FIG. 15A

CONSOL STRAIN GRI-0 TO 25-1 PER PAGE GRI DATA TEMPLATE.GDT 2/6/18



	Location	Sample	Depth, ft	Classification	Initial γ_d, pcf	Initial MC, %
●	B-5	S-3	7.75	Sandy SILT, brown mottled rust, very soft, fine- to coarse-grained sand (Residual Troutdale Formation)	75	42

GRI

CONSOLIDATION TEST

DEC. 2018 JOB NO. 6060 FIG. 16A

CONSOL STRAIN GRI-0 TO 25-1 PER PAGE GRI DATA TEMPLATE.GDT 2/6/18

EMERICK ARCHITECTS

VIEWPOINT INN AND WELLNESS CENTER
ALTERNATIVE PARKING STUDY

Parking for the View Point Inn and Wellness Center will be located on the adjacent property directly to the east, which is under the same ownership as the main property. This location has traditionally been used by the View Point Inn for parking and has previously been permitted and approved as such.

Multnomah County Code, Section 38.4175 lists the following dimensional standards for this property located within the GGF Zone:

- 70% of required off-street parking spaces shall have a minimum width of 9' and minimum length of 18'.
- Up to 30% of the required off-street parking spaces may have a minimum width of 8'-6" and minimum length of 16'.
- Aisle width for 90-degree parking shall be 25'.

The proposal for the View Point Inn will establish an inn, restaurant and wellness retreat center in the historic building. The inn component includes (5) guest rooms for a maximum of (10) overnight guests who will be required to register for a minimum one-week stay. Day visitors will be able to visit the retreat center by appointment only, no drop-ins will be allowed, and they will partake in multiple retreat activities rather than individual appointments. In addition to a limited number of 'by reservation only' commercial events, the restaurant will only be used to serve the overnight guests and day visitors with appointments; it will not be open to the general public.

Per MCC, Section 38.405, the parking layout for the View Point Inn accommodates (27) total spaces, which are distributed as follows:

- (5) for Overnight Accommodations, (12) for Restaurant, (8) for Retreat Facilities and (2) for Residential

Since the restaurant will only be used to serve the overnight guests and day visitors, the required restaurant spaces are redundant to those for the overnight and day uses. Therefore, except for the limited commercial events listed in the Operational Plan, the parking area will not be full most of the time.

The proposed mix of uses for the View Point Inn are indicative of a destination location rather than the typical farm or forest uses found in the GGF zone, and in as such, warrant a variation from the dimensional parking standards of the MCC. By nature, a retreat facility provides a space to withdraw from the day-to-day complications of life, which suggests that most users of the View Point Inn will come to seek respite from urban areas rather than rural ones. Additionally, with an average of (2) persons per overnight room, it is more likely that guests will arrive in smaller, compact vehicles rather than larger vehicles oriented more towards families and groups.

For the above reasons, the proposed parking layout follows the dimensional standards for the City of Portland, which represent a more appropriate guideline for the operations of the View Point Inn and Wellness Center. Portland is the most

EXHIBIT
I.13

prominent urban area that draws tourism to the Gorge and its standards are nationally recognized for their environmentally sensitive approach.

Title 33, Portland Zoning Code, Chapter 33.266, Table 266-4 lists the following dimensional standards for 90-degree parking:

- Stall width = 8'-6"
- Stall depth = 16'
- Aisle width = 20'

Looking beyond Portland, a review of parking standards for other surrounding areas also indicates an alternative to the Multnomah County Standards is warranted for the proposed uses of the View Point Inn.

- Wasco County Land Use & Development Ordinance – Section 20.080 – General Provisions – Off Street Parking and Loading:
  - A parking space is defined as "A minimum gross area available for the parking of a standard American automobile."
  - Access aisles shall be of sufficient width for all vehicle turning and maneuvering.
- Clackamas County Zoning and Development Ordinance – 1015.02 – Parking and Loading – Motor Vehicle Parking Area Standards:
  - Parking spaces shall be a minimum of 8'-6" wide x 16' long.
  - Clackamas County encourages the use of smaller parking stalls by further stating, "A minimum of 25% of required parking spaces shall be no larger than 8'-6" wide and 16' long."
- Camas, WA Code of Ordinances, Chapter 18.11.020 Parking Design:
  - Parking spaces shall be a minimum of 9' wide x 18' long
  - One-way aisles are permitted to be 15' wide

The City of Portland parking standards have been successfully implemented throughout the area and shown to provide a safe parking environment for the proposed operations and anticipated users of the View Point Inn. Additionally, the Portland standards align with the dimensional guidelines set forth by Wasco County, while jurisdictions like Clackamas County show support for smaller parking stall sizes. If it were ascertained that larger parking stall dimensions should be used, then the Camas standards provide an alternative using narrower, one-way drive aisles that could also be executed on the View Point Inn property. The standards of these local jurisdictions provide alternatives to the Multnomah County Code from surrounding areas and have been shown to not create hazardous conditions when put into practice.

Previously approved and permitted parking layouts for this property have featured reduced parking stall and aisle dimensions that do not conform to the MCC dimensional standards. Abiding by the City of Portland standards for the current proposal provides a safe and proven alternative that befits the proposed use of this historic property.

Stormwater Management Facilities

Private Stormwater Report

Viewpoint Inn

HDG Job #: EMA022

Prepared For: HSF LLC

PO Box 70
Corbett, OR 97019

Prepared By:

Humber Design Group, Inc.

117 SE Taylor St. Suite 001
Portland, OR 97214
(P) 503 946 6690

'I hereby certify that this Stormwater Management Report for the Viewpoint Inn project has been prepared by me or under my supervision and meets minimum standards of and normal standards of engineering practice.

REGISTERED PROFESSIONAL ENGINEER 76,071 OREGON JULY 12, 2005 MARTHA WILLIAMSON

EXPIRES 6-30-2018

I hereby acknowledge and agree that the jurisdiction does not and will not assume liability for the sufficiency, suitability, or performance of drainage facilities designed by me.'

Date: January 18, 2018
Revised May 19, 2019

EXHIBIT I.14

Table of Contents

Project Overview and Description

Location of Project	40301 E Larch Mountain Rd, Corbett, OR 97019
Site Area/Acreage	1.21 ac
Nearest Cross Street	NE Columbia Ave, E Larch Mountain Rd
Existing Conditions	Site is currently occupied by decrepit building and poorly maintained gravel driveway and parking areas.
Proposed Development	Proposed development includes renovations to existing historical building, accessory building, refurbishment of existing gravel parking area and paving of Columbia Avenue.
R#	R287200, R287215
Tax Map	1N 5E 30CC
Tax Lot	1500, 1600

Vicinity Map

Site Location

Methodology

Existing Drainage

Runoff from existing building is collected and directs to existing soakage trench at the north of the site. Runoff from Columbia Avenue, which consits of heavily compacted gravel, drains to the north. The remainder of the site, which is undeveloped and pervious, drains steeply to the northwest.

Infiltration Results

Infiltration testing by Geotechnical Resources, Inc. on January 2, 2018 found rates of 3.5 inches per hour at 13.5 feet bgs in the northern portion of the parking lot, and 4 inches per hour at 14 ft bgs in the yard southwest of the house.

PRIVATE Proposed Stormwater Management Techniques

Stormwater runoff from existing building will continue to be managed via existing soakage trench. Runoff from accessory building at the northeast of the site will be collected and piped to a drywell to the north of the existing building.

PUBLIC Proposed Stormwater Management Techniques

The existing drainage pattern of Columbia Avenue will be maintained.

Analysis

Computational Method Used HydroCAD models of a SBUH Type 1A Storm were used to calculate the stormwater management facility sizes for the catchment areas. See attached calculations. Below is a summary of the results.

Hydrologic Soil Group C

Hydrologic Soil Types Mershon silt loam

Table 1 – Curve Numbers

Post-Developed Impervious CN	98

Table 2 – Design Storms

WQ Storm	0.83 inches
2-year	2.40 inches
10-year	3.40 inches
25-year	3.90 inches
100-year	4.40 inches

Table 3 – Time of Concentration

Predeveloped TOC	5 min
Post-Developed TOC	5 min

Table 4 – Catchment Areas and Facility Table

Catchment/ Facility ID	Source (roof, road, etc.)	Treatment Area (sf)	Ownership (private/ public)	Facility Type/ Function	Facility Size
A	Roof	345 sf	Private	Drywell	48" dia x 5' h

Engineering Conclusions

The preceding methodologies and calculations presented indicate compliance with the current jurisdictional stormwater management codes and requirements. A summarized breakdown is presented below:

Water Quality	The proposed development will meet the provisions for water quality per the 2016 Portland Stormwater Management Manual.
Water Quantity	The proposed development will meet the provisions for water quantity per the 2016 Portland Stormwater Management Manual.
Downstream / Upstream Impacts	There are no upstream or downstream impacts created by this proposed development.
100 year storm	The 100 year storm will be safely conveyed away from structures.

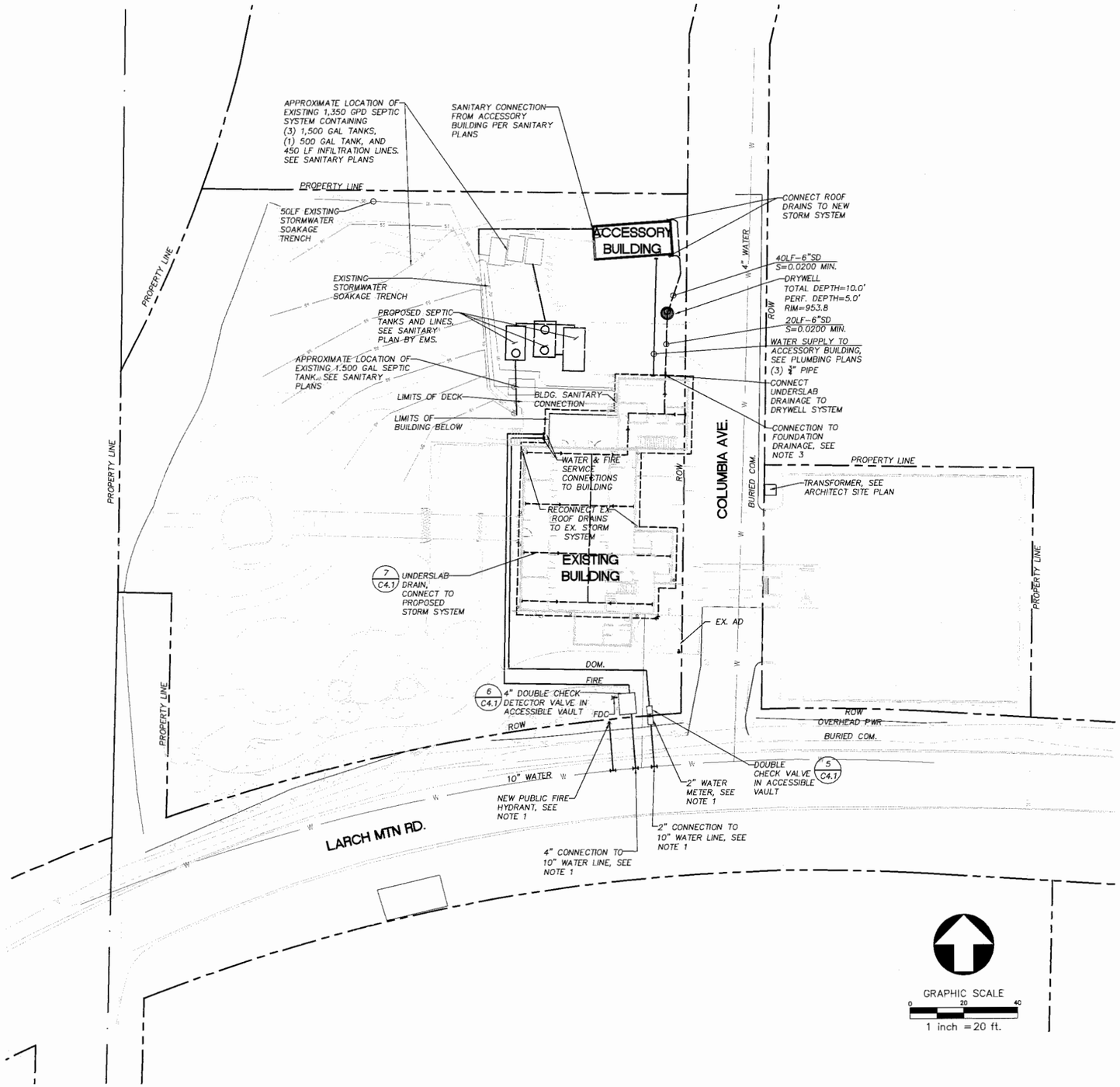
Appendix A

Stormwater Facility Details / Exhibits

Utility Plan

Catchment Map

Drywell Detail



UTILITY NOTES

1. ALL DOMESTIC WATER AND FIRE PROTECTION WORK IN THE PUBLIC RIGHT OF WAY BY CORBETT WATER DISTRICT AT OWNER'S EXPENSE. CONTRACTOR SHALL COORDINATE WITH CORBETT WATER DISTRICT MANAGER, JEFF BUSTO (503)695-2284.
2. EXISTING SEPTIC SYSTEM VAULTS AND PIPING HAVE NOT BEEN SURVEYED. CONTRACTOR SHALL LOCATE EXISTING SEPTIC FACILITIES PRIOR TO CONSTRUCTION TO VERIFY LOCATION. SEE SANITARY PLANS FOR PROPOSED SEPTIC SYSTEM IMPROVEMENTS.
3. INSTALL 4" PERFORATED FOUNDATION DRAIN AROUND PERIMETER OF BUILDING AT FOUNDATION WALL PER DETAIL 6, SHEET C4.1. CONNECT PERFORATED PIPE TO SOLID PIPE WITH CLEANCHECK BACKFLOW PREVENTOR.
4. CAP AND ABANDON EXISTING WATER SERVICE AT MAIN.
5. RECONNECT EXISTING ROOF DRAINS TO EXISTING ROOF DRAINAGE SYSTEM.
6. ALL JOINTS SHALL BE MECHANICALLY RESTRAINED FOR ON-SITE PRESSURE PIPING.

UTILITY LEGEND

- CLEANOUT — 3 / C4.1
- FIRE DEPARTMENT CONNECTION (FDC)
- FIRE HYDRANT
- WATER VAULT
- DRYWELL — 1 / C4.1
- SEPTIC TANK (SEE PLANS BY EMS)

NOTE: INFORMATION IN THESE DOCUMENTS IS NOT APPROVED FOR CONSTRUCTION UNTIL A BUILDING PERMIT HAS BEEN ISSUED.

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Humber Design Group, Inc.
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REV. #: DATE: DESCRIPTION

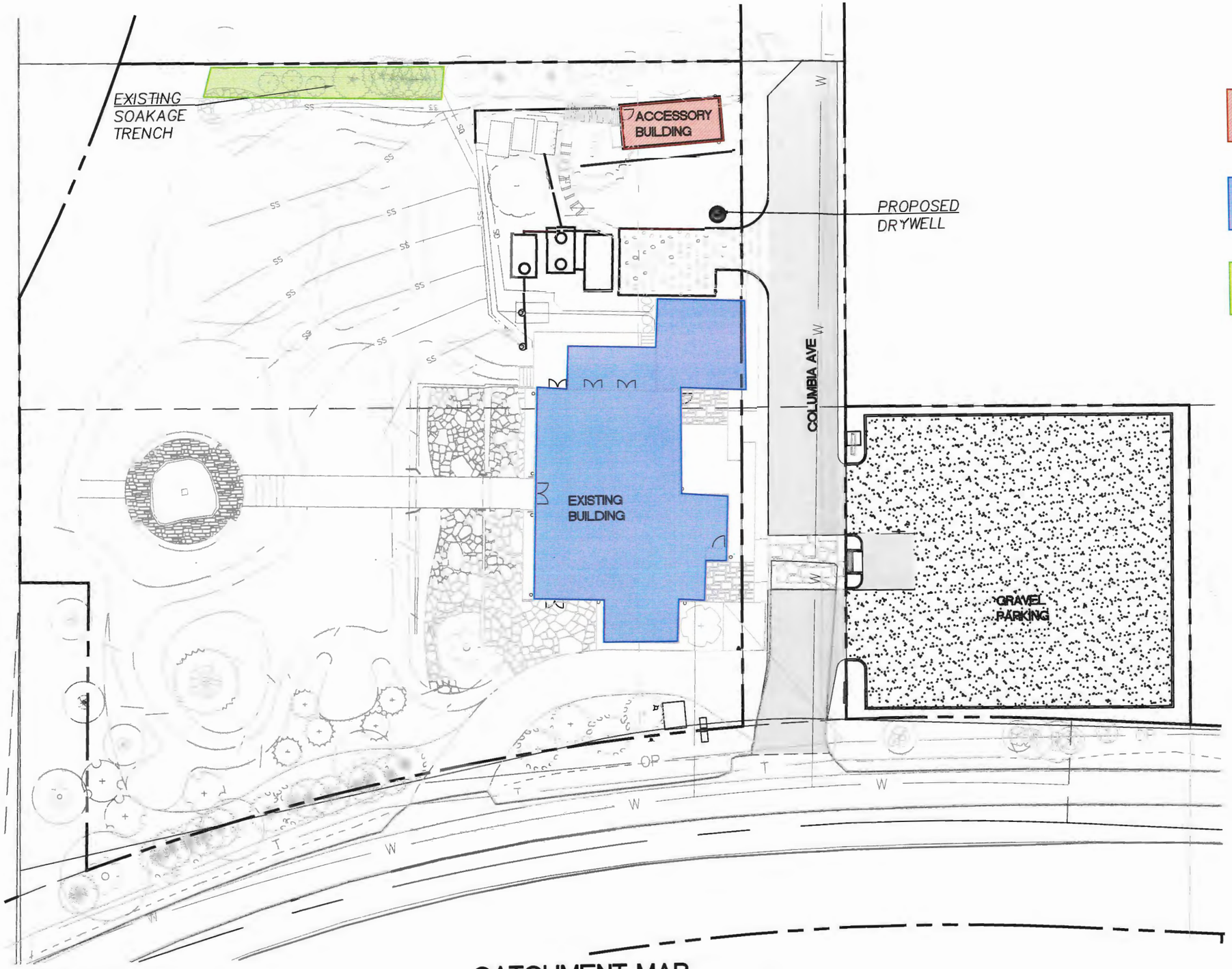
HSF, LLC
VIEW POINT INN & WELLNESS CENTER
40301 E Larch Mountain Rd.,
Corbett, OR 97019

UTILITY PLAN

PROJECT NO: 1723
DRWN BY: PLM | CHK'D BY: MSW
DATE: 05.24.19

C2.0

100% CONSTRUCTION DOCUMENTS



LEGEND

- Proposed Catchment A
  345 sf (to drywell)
- Existing Catchment
  (historic building)
- Existing Soakage Trench

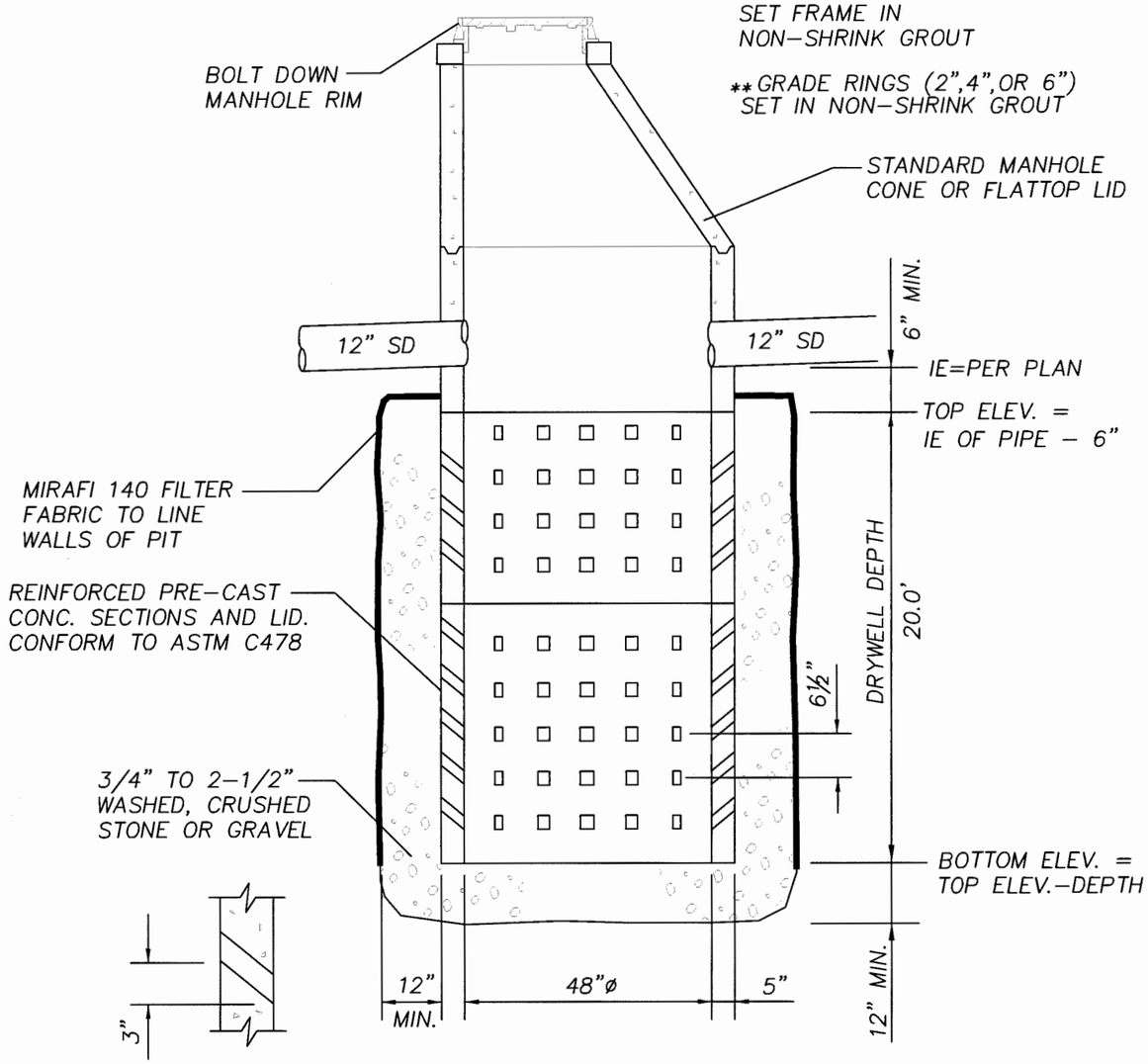
CATCHMENT MAP

1"=30'

GRAPHIC SCALE

0 30 60

1 inch = 30 ft.



DRYWELL

NTS

DRYWELL TESTING NOTES

1. DRYWELL SYSTEMS SHALL HAVE THE CAPACITY TO DISPOSE OF STORMWATER AT THE MINIMUM RATES SPECIFIED BY THE ENGINEER.
2. DRYWELLS SHALL BE TESTED BY THE CONTRACTOR.
3. DRYWELLS SHALL BE TESTED AFTER CONSTRUCTION OF THE DRYWELL STRUCTURE (INCLUDING DRAIN ROCK AND PERIMETER BACKFILL) BUT PRIOR TO THE CONSTRUCTION OF THE TOP SLAB AND FINISH BACKFILL.
4. NOTIFY ENGINEER AND BUREAU OF BUILDINGS INSPECTOR OR AUTHORIZED SPECIAL INSPECTOR 24 HOURS PRIOR TO BEGINNING DRYWELL TESTING.
5. CONTRACTOR SHALL ARRANGE FOR THE PROVISION OF ALL DRYWELL TESTING EQUIPMENT, INCLUDING BUT NOT LIMITED TO FLOW METER (READING IN CUBIC FEET / MINUTE), PIPING, AND TRAFFIC CONTROL.
6. CLEAN WATER SHALL BE PROVIDED TO TEST DRYWELLS, AS APPROVED, INTRODUCTION OF SEDIMENT MAY RESULT IN FAILURE OF THE DRYWELL CAPACITY TEST.
7. FILL DRYWELL WITH WATER AT AN INITIAL RATE OF 100 GPM. EVERY FIVE (5) MINUTES RECORD WATER ELEVATION. MEASURE HEIGHT FROM RIM OF TOP RING TO WATER SURFACE. WHEN WATER SURFACE IN DRYWELL REACHES A CONSTANT ELEVATION, INCREASE FLOW RATE TO 400 GPM AND RECORD ELEVATIONS. CONTINUE TO INCREASE FLOW RATE BY 200 GPM ONCE THE WATER SURFACE RE–STABILIZES, UNTIL CAPACITY IS EXCEEDED OR DRYWELL OPERATES AT 600 GPM, WHICHEVER IS GREATER. MAXIMUM DRYWELL CAPACITY IS THE FLOW RATE FOR THE STABLE CONDITION SET PRIOR TO RATE AT WHICH CAPACITY WAS EXCEEDED.

Appendix B

Support Calculations

HydroCad Report

Summary for Pond 2P: Drywells

Inflow Area = 345 sf,100.00% Impervious, Inflow Depth = 4.16" for 100yr event
Inflow = 0.01 cfs @ 7.90 hrs, Volume= 120 cf
Outflow = 0.00 cfs @ 9.09 hrs, Volume= 120 cf, Atten= 71%, Lag= 71.9 min
Discarded = 0.00 cfs @ 9.09 hrs, Volume= 120 cf

Routing by Stor-Ind method, Time Span= 0.00-30.00 hrs, dt= 0.05 hrs / 2
Peak Elev= 100.94' @ 9.09 hrs Surf.Area= 38 sf Storage= 19 cf

Plug-Flow detention time= 58.2 min calculated for 120 cf (100% of inflow)
Center-of-Mass det. time= 58.1 min (716.7 - 658.6)

Volume	Invert	Avail.Storage	Storage Description
#1	100.00'	63 cf	**4.00'D x 5.00'H Vertical Cone/Cylinder**Inside #2
#2	100.00'	39 cf	**7.00'D x 5.00'H Vertical Cone/Cylinder** 192 cf Overall - 63 cf Embedded = 130 cf x 30.0% Voids
		102 cf	Total Available Storage

Device	Routing	Invert	Outlet Devices
#1	Discarded	100.00'	**1.750 in/hr Exfiltration over Wetted area**

Discarded OutFlow Max=0.00 cfs @ 9.09 hrs HW=100.94' (Free Discharge)
1=Exfiltration (Exfiltration Controls 0.00 cfs)

Pond 2P: Drywells