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August 30, 2013 W1115 GEOTECHNICAL RPT

Kennedy/Jenks Consultants
32001 32nd Avenue S., Suite 100
Federal Way, WA 98001

Attention: Ty Schreiner

SUBJECT: Geotechnical Investigation
Sheet Pile Bulkhead Wall
Cornet Bay Marina
Oak Harbor, Washington

At your request, GRI has conducted a geotechnical investigation for the above-referenced project at the Cornet Bay Marina in Oak Harbor, Washington. The Vicinity Map, Figure 1, shows the general location of the site. The purpose of the investigation was to evaluate subsurface conditions at the site and develop recommendations for design and construction of temporary shoring and a proposed bulkhead wall. The investigation included a review of existing subsurface information for the site, subsurface explorations, laboratory testing, and engineering analyses. This report describes the work accomplished and provides our conclusions and recommendations for the design and construction of the proposed bulkhead wall.

As part of our investigation, GRI reviewed the following reports previously prepared by others:

> "Screening Survey for Petroleum Contamination at Cornet Bay Marina (Island County)," dated September 2005, prepared by the Washington State Department of Ecology (WSDOE).
>
> "Cornet Bay Marina Bulkhead Assessment," dated November 6, 2006, prepared by Reid Middleton, Inc.

PROJECT DESCRIPTION

The site is located on Cornet Bay at the northern end of Whidbey Island in Oak Harbor, Washington. As shown on the Site Plan, Figure 2, an existing 12-ft-tall timber bulkhead wall is located at the edge of the marina shoreline. Lateral support for the existing bulkhead wall is provided by cable tie-backs installed in the backfill behind the bulkhead. We understand there was a release of gasoline and/or diesel fuel from a tank located behind the existing bulkhead, and the Washington State Department of Ecology (Ecology) is requiring removal of a significant amount of the contaminated soil. Our review of preliminary plans developed by Kennedy/Jenks Consultants (KJ) indicates the required excavation will extend up to approximately 120 ft (horizontal distance) behind the existing bulkhead. The excavation will extend to a maximum depth of approximately 18 ft and will typically be less than about 12 ft deep.

As currently planned, a shoring system consisting of cantilevered, tight-joint sheet piles will be constructed on the water side of the existing bulkhead to facilitate the required temporary excavation and removal of the existing timber bulkhead, which is near the end of its useful life. Following removal of the contaminated soils and the existing timber bulkhead, the sheet piles will remain in place to provide permanent lateral support for the new bulkhead. To provide additional lateral resistance, we understand a mechanically stabilized earth (MSE) retaining wall will be constructed in the backfill area behind the sheets. The maximum height of the finished bulkhead will be about 13 ft above the mudline elevation.

Our scope of work for this project includes performing subsurface investigations at the site, completing a laboratory testing program, and completing engineering analyses that lead to recommendations regarding design and construction of a sheet pile bulkhead wall.

SITE DESCRIPTION

Topography and Existing Conditions

Survey data provided by KJ indicate the ground surface behind the bulkhead is relatively flat at about elevation 14 to 16 ft (NAVD 88). The mudline at the Cornet Bay Marina is near elevation 1 ft and is exposed during low tide. Tidal fluctuations at the site range up to about 13 ft from approximately elevation -1 to 12 ft (MLLW).

The area behind the bulkhead is typically surfaced with gravel and low grass with an area of Portland cement concrete pavement around the existing building and entrance to the marina pier. The building is used as the marina store and office and is located near the center of the site; the gravel areas are used for parking.

Geology

Subsurface explorations completed by GRI and KJ indicate the site is mantled with 10 to 15 ft of fill that primarily consists of clay, silt, sand, and gravel. The fill soils are underlain by Pleistocene glacial drift of the Vashon Stade, which consists of bedded clay, silt, and sand units of varying thicknesses (Pessl, et al., 1989).

SUBSURFACE CONDITIONS

General

Subsurface materials and conditions at the site were evaluated between June 20 and 21, 2013, with three borings, designated B-1 through B-3. The borings were advanced to depths of about 38 to 80 ft at the approximate locations shown on Figure 2. The borings were completed using hollow-stem auger techniques to a depth of about 20 ft and mud-rotary methods below this depth. Details of the field and laboratory testing programs completed for this investigation are provided in Appendix A. Logs of the borings are provided on Figures 1A through 3A. The terms used to describe the materials encountered in the borings are defined in Table 1A.

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

1. **FILL**
2. **SAND**
3. **CLAY**
4. **SILT and SAND**

1. FILL. Fill consisting of silt and sand was encountered at the ground surface in all borings completed at the site and extends to depths of 11.5 to 12.5 ft. A strong petroleum odor was noted in the fill in boring B-3. The silt fill is typically gray and contains a variable sand and clay content, ranging from some fine-grained sand to sandy and a trace to some clay. The sand fill is typically gray, fine grained, and contains a variable silt and clay content ranging from trace to some silt and clay. Scattered gravel and organics are present in the fill. Shell fragments were observed near the transition between the fill and underlying native materials. Based on Standard Penetration Test (SPT) N-values of 2 to 6 blows/ft, the relative consistency of the silt fill ranges from soft to medium stiff, and the relative density of the sand fill ranges from very loose to loose. The natural moisture content of the fill ranges from about 10 to 27%.

2. SAND. Sand was encountered beneath the fill in boring B-1 and extends to a depth of 17 ft. The sand is typically gray, fine grained, silty, and contains some clay and scattered shells and gravel. Based on N-values of 5 and 18 blows/ft, the relative density of the sand ranges from loose to medium dense. The natural moisture content of the sand ranges from about 27 to 36%.

3. CLAY. Clay was encountered beneath the fill soils in borings B-2 and B-3 and beneath the sand in boring B-1. The clay is typically gray and contains a variable sand and silt content, ranging from a trace of fine- to medium-grained sand to sandy and trace of silt to silty. Scattered gravel and shell fragments are present in the clay. Based on N-values of 0 to 33 blow/ft, the relative consistency of the clay ranges from very soft to hard and is typically medium stiff. The upper surface of the clay unit is typically stiff, and zones of very soft to soft clay were encountered at depths of 30 and 35 ft. Atterberg limits testing indicates the clay has a low to medium plasticity, see Figure 4A. The natural moisture content of the clay ranges from about 19 to 41%. Boring B-2 was terminated in clay at a depth of 38 ft.

4. SILT and SAND. Interbedded layers of sand and silt were encountered beneath the clay in borings B-1 and B-3 and extend to a depth of about 80 ft in boring B-1, the maximum depth explored. The thickness of the interbedded layers encountered in the borings range from 5 to 25 ft. The sand is typically gray, fine grained, contains a variable silt and clay content, ranging from some silt to silty and trace to some clay, and contains scattered gravel. The silt is typically gray and contains a variable sand and clay content, ranging from some fine-grained sand to sandy and trace to some clay. Based on N-values of 14 to more than 50 blows/ft, the relative density of the sand ranges from medium dense to very dense. Based on N-values of 8 to more than 50 blows/ft, the relative consistency of the silt ranges from medium stiff to hard. The natural moisture content of the sand ranges from about 15 to 27%. The natural moisture content of the silt ranges from about 19 to 38%. Borings B-1 and B-3 were terminated in the silt and sand at depths of 80.3 and 51.5 ft, respectively.

Groundwater

The borings were advanced using hollow stem auger and mud-rotary drilling methods, which do not allow measurement of groundwater levels during drilling. Review of WSDOE water well logs indicates the depth to groundwater at the site is on the order of 2 to 12 ft below the ground surface. The groundwater level at the site will vary in response to tidal fluctuations and will likely be near the level of the water in the bay. In addition, groundwater from the upland areas to the southeast will affect groundwater levels in the area. Perched groundwater conditions may develop near the ground surface during periods of prolonged or intense precipitation.

CONCLUSIONS AND RECOMMENDATIONS

General

The explorations completed for the project indicate the site is typically mantled with relatively soft or loose silt and sand fill to a maximum depth of about 12.5 ft. The fill soils are underlain by relatively stiff to dense clay, silt, and sand soils of glacial origin. The groundwater level at the site will closely reflect the water level in Cornet Bay and will fluctuate with tidal variations and may approach the ground surface during high tides or during the wet, winter months. Recharge from the upland areas southeast of the site will likely also affect groundwater conditions.

In our opinion, important geotechnical-related aspects of the project include temporary shoring, excavation, dewatering, and the loose, moisture-sensitive fill soils present to the planned depths of excavation. Temporary excavations on the order of 10 ft deep will be required to remove the contaminated soils from behind the bulkhead wall. The bottom of the excavations will extend below groundwater levels, particularly during high tides. The following sections of this report provide our conclusions and recommendations for design and construction of the proposed shoring and bulkhead wall.

Temporary Shoring and Permanent Bulkhead Wall

As currently planned, cantilevered, tight-joint, driven steel sheet piles will be used to support the required temporary excavation behind the existing bulkhead wall. Following excavation and backfilling of the site, the sheet piles will also serve as the permanent bulkhead wall. To develop stability, the sheet piles will be driven below the bottom of the excavation and could be designed to reduce seepage into the temporary excavation and dewatering quantities. Based on preliminary plans provided by KJ, the sheet piles will be driven to a tip elevation of approximately -33 ft. In our opinion, the critical water levels for the stability of the temporary shored excavation will occur at high tide, when the maximum hydrostatic pressure of the bay is acting on the wall, and the excavation is completed to design grade. Lateral earth pressure criteria for design of the sheet pile wall during temporary excavation conditions are provided on Figure 3. Lateral earth pressure criteria for design of the sheet pile wall for the finished conditions are provided on Figure 4. Additional lateral pressures induced by surcharge loads can be estimated using the guidelines provided on Figure 5.

Excavation and Groundwater Control

The method of excavation and design of the temporary shoring and groundwater management system is the responsibility of the contractor. The means, methods, and sequencing of construction operations and site safety are also the responsibility of the contractor. We recommend the contractor submit for review an excavation and dewatering plan prepared by a professional engineer registered in Washington. 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, site safety, or design.

The borings completed at the site encountered fill consisting of silt and sand with scattered gravel and organics. These soils can be readily excavated using conventional equipment. However, some riprap slope protection was observed adjacent to the northwest bulkhead return wall, and larger cobbles, boulders, and construction debris could be encountered in the fill soils. The presence of these materials could require larger excavation equipment and a specialized shoring and dewatering plan specific to those conditions.

The sand fill and underlying native sand may yield substantial inflows of groundwater into the excavations. Significant seepage into excavations in silt and clay can occur through preferential paths consisting of fine voids or holes. It should be noted that during the drilling of boring B-2, a significant loss of drilling fluid occurred at a depth of about 38 ft, and drilling fluid was observed on the exposed mudline below the bulkhead, indicating the presence of a preferential path between the bay and the area behind the bulkhead wall. However, in general, the silt and clay is much less permeable than the sand, and the installation of sheet piles will significantly reduce any seepage through preferential paths that may be present within the depth of the sheets. Positive control of groundwater will be an important consideration during excavation and backfilling activities, particularly in the location of the MSE wall. The dewatering system should be capable of maintaining the groundwater level at a minimum depth of 2 ft below the base of the excavation, or as required to maintain a stable excavation bottom. Control of groundwater will depend on the materials and groundwater levels encountered in the excavation and the contractor's approach to the work. It should be anticipated that dewatering by wells or well points will be necessary if an open cut is used. We anticipate that it may be necessary to complete the excavation in small sections to limit the dewatering quantities that will require treatment and disposal.

We anticipate that side slopes can be excavated to a maximum inclination of about 1.5H:1V following dewatering. Without a dewatering system, the sand fill will tend to cave and "run," forming very flat slopes. It may be feasible to control the inflow of groundwater with ordinary sump pumping if sheet pile shoring is used instead of an open cut.

MSE Wall Design

As currently planned, an MSE retaining wall will be constructed in the excavated area behind the steel sheet pile wall. We understand the purpose of the MSE retaining wall is to provide additional resistance to lateral loading behind the permanent bulkhead. Based on our discussions with KJ, we understand the MSE wall could be designed to range in height from 3 to 12 ft. Design of the MSE wall will be completed by others, and detailed design information is not currently available.

To facilitate drainage of the reinforced and retained zones and limit lateral deformations, we recommend using relatively clean, granular backfill in the reinforced zone of the wall. Sand, sand and gravel, or crushed rock with less than 5% passing the No. 200 sieve (washed analysis) would be appropriate for this purpose. We anticipate the general wall backfill behind the reinforced zone will consist of native on-site soils and imported granular fill consisting of sand, sandy gravel, or similar materials.

Proper drainage is an essential part of retaining wall design and will be particularly important at this site due to high groundwater levels and tidal fluctuations. We recommend constructing a chimney drain between the MSE wall backfill and the fascia and also between the MSE reinforced zone fill and general backfill. In addition, a drainage blanket should be installed at the base of the reinforced zone to connect the two chimney drains. In our opinion, an 18-in.-wide chimney drain (vertical drainage blanket) will be suitable if the fill in the reinforced zone consists of relatively clean granular material that contains less than about 5% passing the No. 200 sieve (washed analysis). The drainage blankets should consist of crushed rock that conforms to WSDOT Standard Specifications Section 9-03.12(4) Gravel Backfill for Drains. Recommended drainage details are shown on Figure 6. We have assumed the design will incorporate sufficient drainage of the chimney drains and drainage blanket through the wall fascia to provide essentially drained backfill conditions. However, to account for partially drained conditions in the retained zone due to high groundwater conditions and fluctuating tides, we recommend designing for a minimum 5-ft height of hydrostatic water pressure above the bay mudline. The design should assume submerged conditions for the MSE foundation soils. The following table summarizes our recommended soil parameters for design of the MSE wall.

Soil Type	Soil Properties				
	Total Unit Weight, γ_T, pcf	Buoyant Unit Weight, γ', pcf	ϕ'	c, psi	Ka
Native Sand and Clay (Foundation Soil)	128	66	30°	0	NA
Free-Draining Granular Structural Fill (Reinforced Zone)	130	68	34°	0	0.28
Drain Rock (Drainage Blanket)	130	68	34°	0	0.28
On-Site Structural Fill (Retained Zone)	128	66	32°	0	0.30

All fill should be compacted as structural fill to 95% of the maximum dry density determined in accordance with ASTM D 698.

Additional lateral load due to seismic forces on retaining walls can be evaluated based on a triangular lateral earth pressure distribution with a maximum pressure of 14H at the ground surface and 0 at the base of the wall, where H is the height of the wall. The resultant force acts at a point above the base of the wall equal to 60% of the wall height. Additional lateral pressures induced by surcharge loads can be estimated using the guidelines provided on Figure 5. New foundations should not be located within the limits of the reinforced zone or within the active zone located behind the reinforced zone. We recommend that new foundations be set back at least 1.5H from the reinforced zone, where H is the total wall height.

The MSE foundation subgrade soils consist of silty sand, silt, and clay soils that are moisture sensitive and will be easily disturbed by construction actives. Therefore, the contractor should use construction equipment and procedures that minimize disturbance and softening of the subgrade soils, particularly if wet conditions are encountered. Subgrade soils that are disturbed or softened during construction should be overexcavated and backfilled with compacted granular structural fill. The soil properties we have recommended for design of the MSE wall are based on foundation soils that are firm and undisturbed. In our opinion, the MSE wall foundation subgrade should be evaluated by a qualified geotechnical engineer, and any areas of soft subgrade, loose fill, or other unsuitable material should be overexcavated and replaced with compacted crushed rock. We anticipate the governing design codes will require a minimum wall embedment of at least 2 ft based on the sloping mudline conditions in front of the wall.

Construction of the MSE wall and backfill will induce consolidation of the underlying silt and clay soils and settlement at the ground surface. Based on the anticipated depth of excavation required to remove the contaminated soils and establish the base of the MSE wall, the underlying soils will experience unloading and re-loading during construction, which will help reduce the total amount of primary consolidation settlement. We anticipate the majority of the consolidation settlement will occur during placement of the MSE wall backfill, and consolidation will be essentially complete within 1 month after the fill has been placed.

Structural Fill

All new fill in structural areas should be compacted as structural fill. In our opinion, on-site soils that are free of organics and other deleterious materials and debris are suitable for use in structural fills. As noted above, it should be anticipated that near-surface, silty soils will be encountered locally. Silty soils are sensitive to moisture content and can be placed and adequately compacted only during the dry, summer months. Fills constructed in wet conditions, fills should be constructed using imported granular materials that are relatively clean.

In general, approved on-site or imported, organic-free, fine-grained sand and silty soils used to construct structural fills within areas of mass filling, structures, and pathways should be placed in 9-in.-thick lifts (loose) and compacted using medium-size (48-in.-diameter), segmented-pad or vibratory rollers to a density not less than 95% of the maximum dry density as determined by ASTM D 698. Pieces of rock or concrete larger than about 6 in. should be removed from the fill prior to compaction. Fill placed in landscaped areas should be compacted to a minimum of about 90% of the maximum dry density as determined by ASTM D 698. In our opinion, the moisture content of silty sand, silt, and clay soils at the time of compaction should be controlled to within 3% of optimum. Some moisture conditioning of silty sand, silt, and clay soils may be required to achieve the recommended compaction criteria. All structural fills should extend a minimum horizontal distance of 5 ft beyond the limits of building and pavement areas.

On-site or imported granular material used to construct structural fills or work pads during wet weather can consist of relatively clean granular material, such as sand, sand and gravel, or crushed rock with a maximum size of about 4 in. and with not more than about 5% passing the No. 200 sieve (washed analysis). The first lift of granular fill material placed over silt subgrade should be in the range of 12 to 18 in. thick (loose). Subsequent lifts should be placed 12 in. thick (loose). All lifts should be compacted to at least 95% of the maximum dry density as determined by ASTM D 698 using a medium-weight (48-in.-diameter drum), smooth, steel-wheeled, vibratory roller. Generally, a minimum of four passes with the roller are required to achieve compaction.

Depending on actual soil and groundwater conditions at the time of construction, we anticipate it will be necessary to overexcavate the subgrade to allow installation of bottom stabilization material to provide a relatively firm base and facilitate dewatering of the excavations by pumping with sumps. The actual amount of overexcavation will need to be evaluated on the basis of field observations made during construction; however, we anticipate approximately 1 to 2 ft of stabilization material may be needed. The bottom stabilization material should consist of clean, well-graded crushed rock with a maximum size of about 4 in. and less than 2% passing the No. 200 sieve (washed analysis). The stabilization rock should be placed in a single lift and tamped into place until well-keyed using hand compaction equipment. If needed the stabilization material may be capped with about 6 in. of compacted 3/4-in.-minus crushed rock to serve as a leveling course and choke off the surface of the coarser-graded stabilization material.

Seismic Considerations

We understand the project will be designed in accordance with the 2012 International Building Code (IBC). Seismic design in accordance with the 2012 IBC is based on the ASCE 7-10 document. The IBC design methodology uses two spectral response coefficients, S_S and S_1, corresponding to periods of 0.2 and 1.0 second, to develop the design-level earthquake spectrum. The S_S and S_1 coefficients for the site located at the approximate latitude and longitude coordinates of 48.40°N and 122.63°W are 1.21 and 0.48 g, respectively. The site is designated Site Class E based on the estimated SPT N-value profile for the upper 100 ft in accordance with Chapter 20 of ASCE 7-10. We recommend using the Site Class E designation for design of the project.

Based on the relative density and plastic fines content of the sandy soils at the site, it is our opinion the risk of widespread liquefaction and lateral spreading is low. There is some risk of liquefaction in isolated, discontinuous zones of looser, lower-plasticity sands at the site; however, we do not anticipate significant settlement or lateral spreading will occur as a result of isolated liquefaction. Based on our review of available geologic information for the project area, the risk of earthquake-induced fault displacement at the site is very low, unless occurring on a previously unmapped fault. The USGS deaggregations for the site (USGS, 2013) indicate several faults are present in the project area. The risk of damage by tsunami and/or seiche at the site is present.

Design Review and Construction Services

The contractor is responsible for design of temporary excavation shoring and dewatering. In this regard, the shoring and dewatering plans should be designed and stamped by a licensed engineer and submitted to the design team for review. This report addresses geotechnical considerations regarding the general approach to design and construction of the shoring and dewatering and is for informational purposes only. The information in this report should not be interpreted to mean that GRI is providing design of the shoring and dewatering systems, which is solely the responsibility of the contractor.

We welcome the opportunity to review and discuss construction plans and specifications for this project as they are being developed 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 and the plans and specifications, we are of the opinion that all construction operations dealing with earthwork and shoring 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 this 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 in the design of the 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 design and construction of the excavations and shoring. 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 our recommendations in writing.

The analyses and recommendations submitted in this report are based on the data obtained from the borings recently made at the site, laboratory test results, and 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 and that groundwater levels will fluctuate with time. This report does not reflect any variations that may occur between these explorations. The nature and extent of variations may not become evident until construction. If, during construction, subsurface conditions different from those described in this report are observed or encountered, we should be advised at once so that we can observe these conditions and reconsider our recommendations where necessary.

Please contact the undersigned if you have any questions regarding this report.

Submitted for GRI,

MATTHEW S. SHANAHAN
STATE OF WASHINGTON
44077
REGISTERED
PROFESSIONAL ENGINEER

Expires 4/2014

Matthew S. Shanahan, PE
Associate

Michael W. Reed, PE
Principal

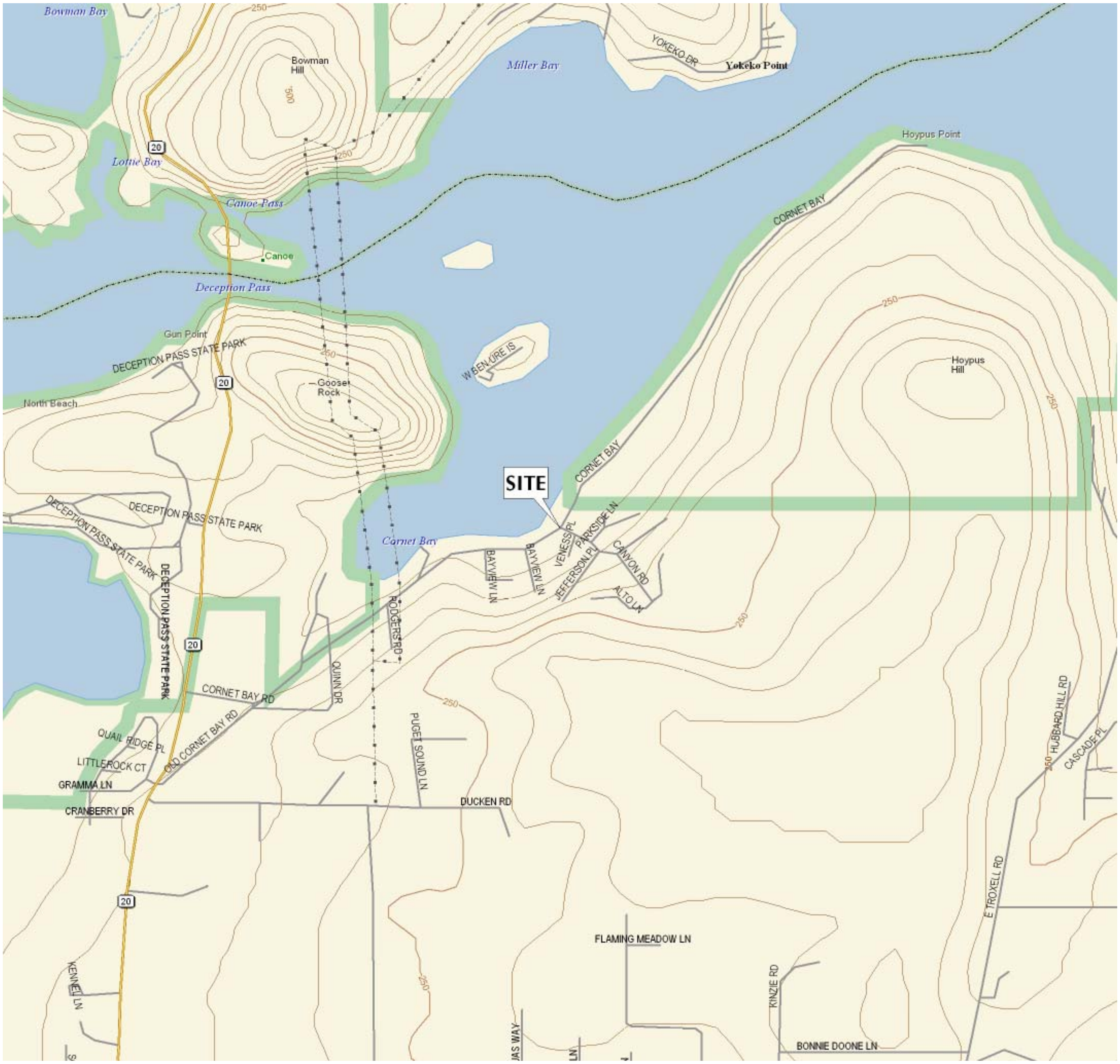
John K. (Jack) Gordon, PE
Project Engineer

This document has been submitted electronically.

References

Pessl, F., Dethier, D.P., Booth, D.B., and Minard, J.P., Surficial geologic map of the Port Townsend 30- by 60-minute quadrangle, Puget Sound region, Washington: U.S. Geological Survey, Miscellaneous Investigations Series Map I-1198-F

U.S. Geological Survey, 2013, Probabilistic hazard lookup by latitude, longitude, accessed 7/11/13, from USGS website: https://geohazards.usgs.gov/deaggint/2008/



DELORME 3-D TOPOQUADS, WASHINGTON
DECEPTION PASS, WASH. (4da) 2004

North

0 1/2 1 MILE

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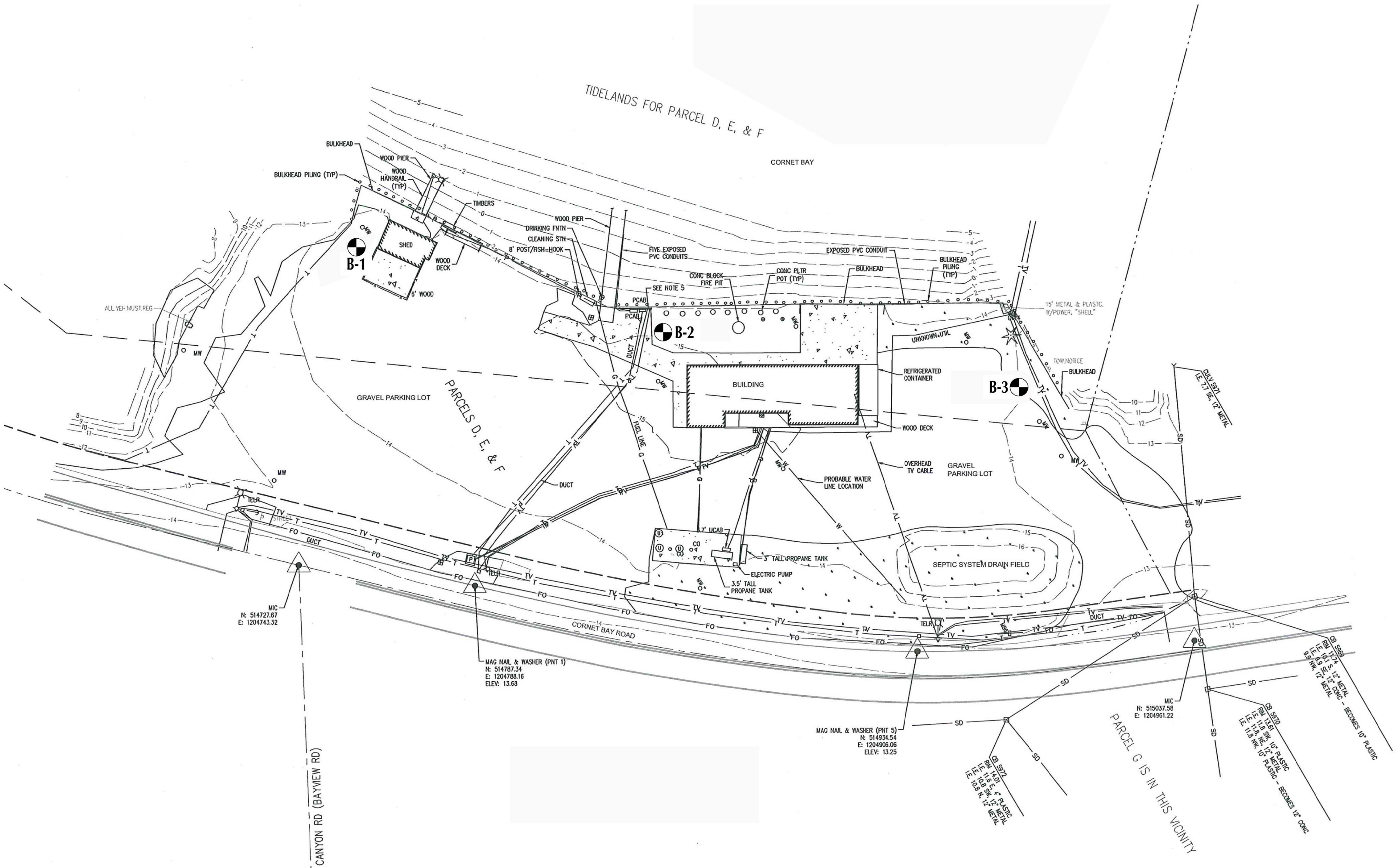
CORNET BAY MARINA

VICINITY MAP

AUG. 2013 JOB NO. W1115 FIG. 1



DATUM AND ROW NOTES

1. BASIS OF BEARINGS IS WASHINGTON STATE PLANE COORDINATE SYSTEM, NORTH ZONE, NORTH AMERICAN DATUM OF 1983, 1991 ADJUSTMENT (NAD 83/91).
2. THE VERTICAL DATUM IS THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD 88), AS DERIVED FROM GPS OBSERVATIONS.
3. BOUNDARY ESTABLISHED PER RECORD OF SURVEY FILED IN VOLUME 7 OF SURVEYS, PAGES 233 AND 234, RECORDING NO. 91001920, RECORDS OF ISLAND COUNTY, WA.
4. HELD FOUND MONUMENTS AT THE INTERSECTION OF CORNET BAY ROAD WITH BAY VIEW ROAD AND AT THE NORTH QUARTER CORNER OF SECTION 36.
5. PARCELS A-G ARE PER QUIT CLAIM DEED RECORDED IN BOOK 811, PAGES 2437-2449, RECORDS OF ISLAND COUNTY, WA.

BORING MADE BY GRI (JUNE 19 - 21, 2013)

SITE PLAN FROM 50% SUBMITTAL PLAN SET BY KENNEDY / JENKS CONSULTANTS

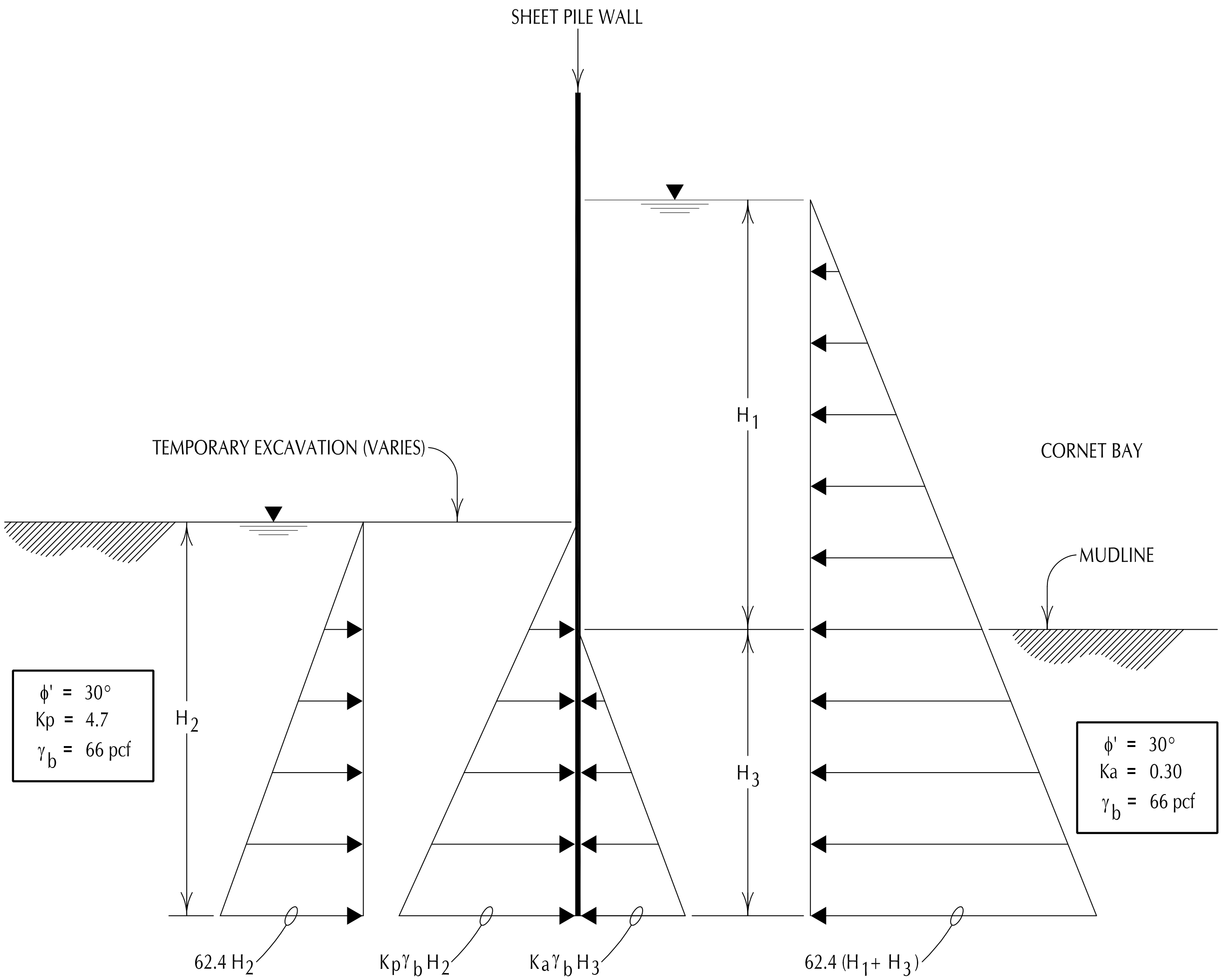
North

0 50 100 FT

GRI KENNEDY / JENKS CONSULTANTS
CORNET BAY MARINA

SITE PLAN

AUG. 2013 | JOB NO. W1115 | FIG. 2



NOTES:

1) EARTH PRESSURES SHOWN ARE FOR CRITICAL WATER LEVELS AT HIGH TIDE.

2) EARTH PRESSURES ARE UNFACTORED, AND PASSIVE RESISTANCES HAVE NOT BEEN DECREASED TO LIMIT WALL DEFORMATION.

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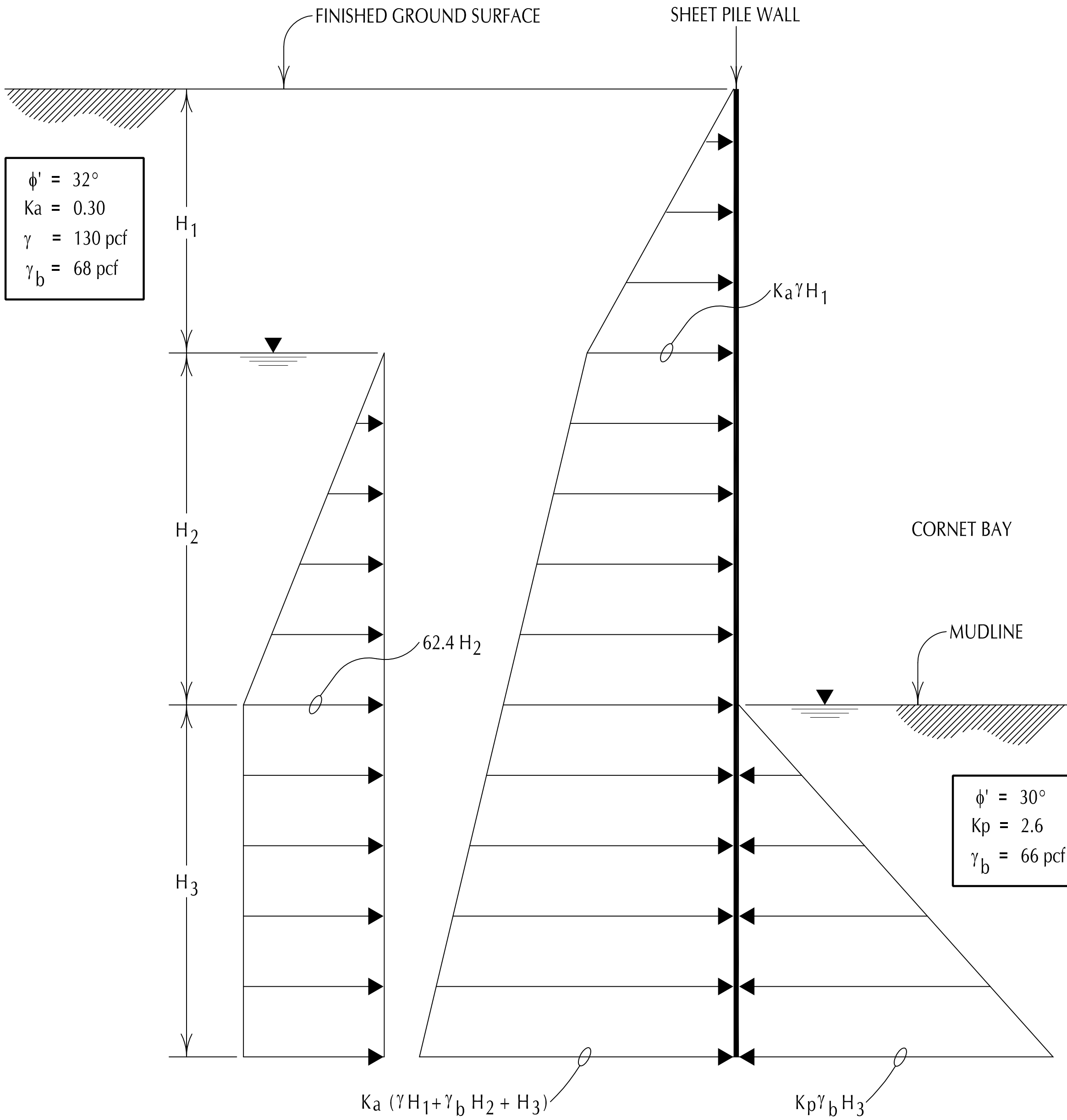
LATERAL EARTH PRESSURES FOR TEMPORARY EXCAVATION

AUG. 2013 | JOB NO. W1115 | FIG. 3



NOTES:

1) ASSUMES EXCAVATION WILL BE BACKFILLED WITH RELATIVELY CLEAN GRANULAR STRUCTURAL FILL.

2) PASSIVE EARTH PRESSURE IS LOWEST FOR BAY LEVEL AT OR ABOVE MUDLINE.

3) EARTH PRESSURES ARE UNFACTORED, AND PASSIVE RESISTANCE HAS NOT BEEN DECREASED TO LIMIT WALL MOVEMENT.

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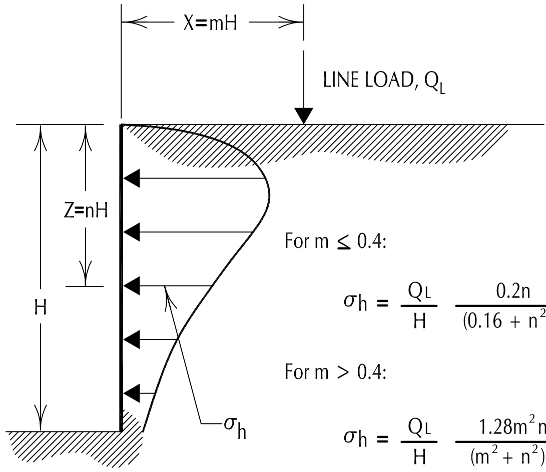
LATERAL EARTH PRESSURES FOR FINISHED CONDITIONS

AUG. 2013 | JOB NO. W1115 | FIG. 4



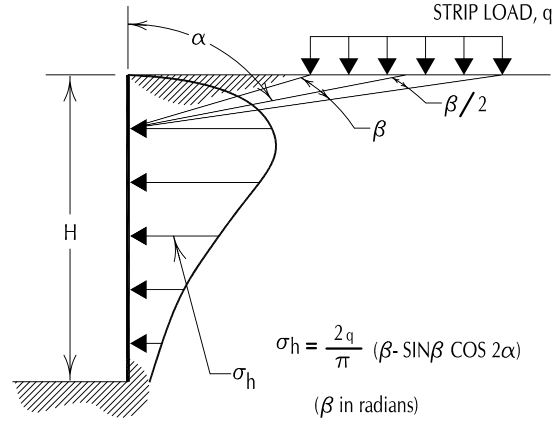
For m ≤ 0.4:

$$\sigma_h = \frac{Q_L}{H} \frac{0.2n}{(0.16 + n^2)^2}$$

For m > 0.4:

$$\sigma_h = \frac{Q_L}{H} \frac{1.28m^2 n}{(m^2 + n^2)^2}$$

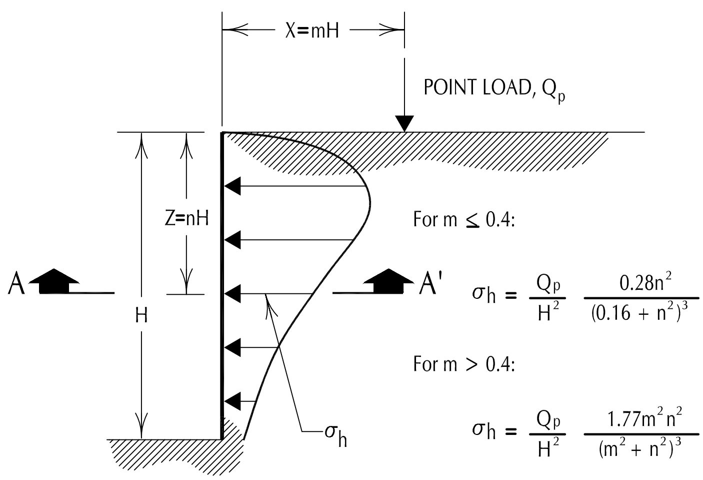
LINE LOAD PARALLEL TO WALL



$$\sigma_h = \frac{2q}{\pi} (\beta - \text{SIN}\beta \text{ COS } 2\alpha)$$

(β in radians)

STRIP LOAD PARALLEL TO WALL

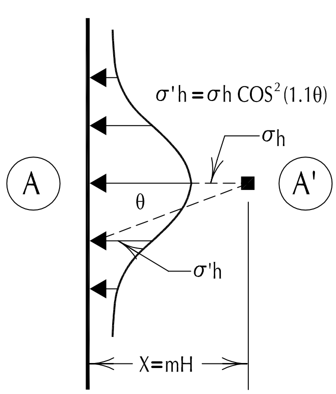


For m ≤ 0.4:

$$\sigma_h = \frac{Q_p}{H^2} \frac{0.28n^2}{(0.16 + n^2)^3}$$

For m > 0.4:

$$\sigma_h = \frac{Q_p}{H^2} \frac{1.77m^2 n^2}{(m^2 + n^2)^3}$$



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.

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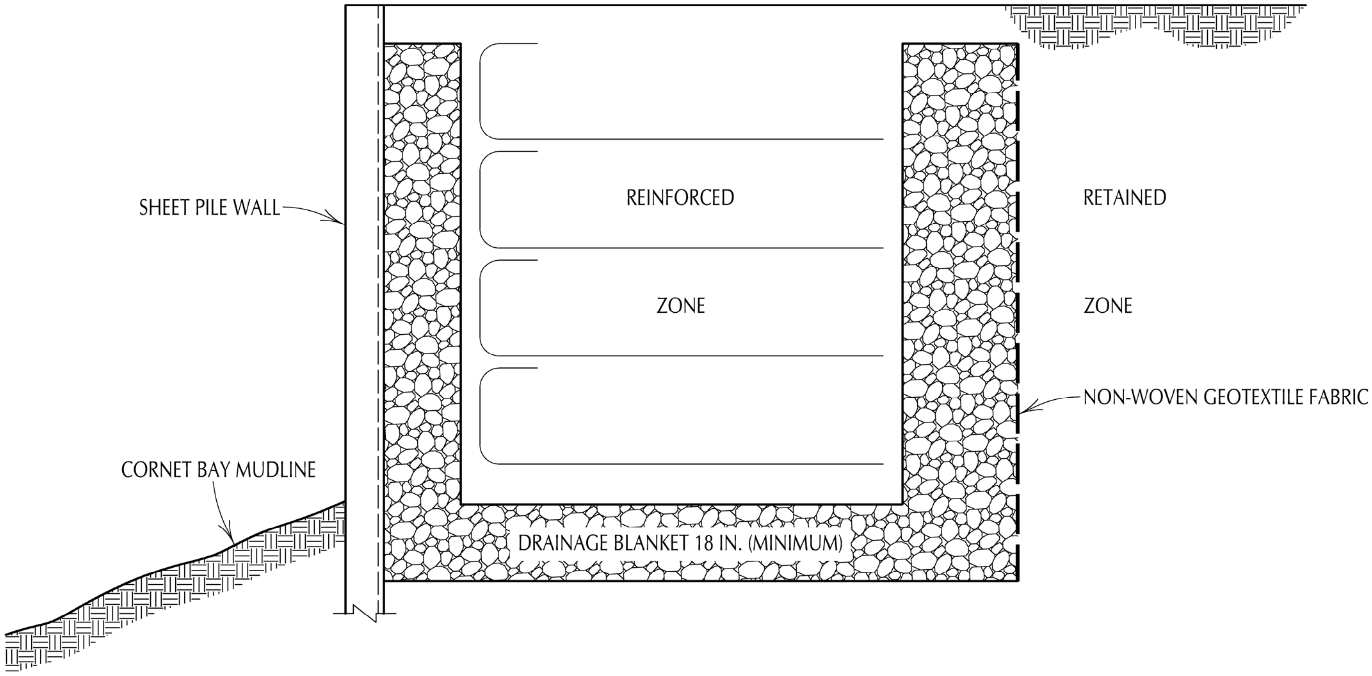
SURCHARGE-INDUCED LATERAL PRESSURE

AUG. 2013 | JOB NO. W1115 | FIG. 5



SOIL PROPERTIES FOR MSE WALL DESIGN

CORNET BAY MARINA REMEDIATION (GRI #W1115)

		Soil Properties				
Soil Type	**γ, pcf**	**γ', pcf**	**ϕ'**	**c, psi**	**Ka**
Native Sand & Clay (Foundation Soil)	128	66	30°	0	NA
Compacted Crushed Rock (Reinforced Zone)	130	68	34°	0	NA
Compacted Crushed Rock (Retained Zone)	130	68	34°	0	0.28
Compacted Native Fill (Retained Zone)	128	66	32°	0	0.30

NOTES:

1) THE SOIL PROPERTIES ASSUME THE MSE WALL IS DESIGNED TO PROVIDE ESSENTIALLY DRAINED CONDITIONS. HOWEVER, TO ACCOUNT FOR PARTIALLY DRAINED CONDITIONS IN THE RETAINED ZONE DUE TO HIGH GROUNDWATER AND TIDAL FLUCTUATIONS, WE RECOMMEND DESIGNING FOR A MINIMUM 5-FT HEIGHT OF HYDROSTATIC WATER PRESSURE ABOVE THE BAY MUDLINE.

2) GRANULAR STRUCTURAL FILL WITH LESS THAN 5% PASSING THE NO. 200 SIEVE (WASHED ANALYSIS) RECOMMENDED FOR THE REINFORCED ZONE.

3) DRAIN ROCK SHOULD CONFORM TO WSDOT STANDARD SPECIFICATIONS 9-03.12(4) GRAVEL BACKFILL FOR DRAINS.

4) ALL MSE WALL BACKFILL SHOULD BE COMPACTED AS STRUCTURAL FILL TO 95% OF THE MAXIMUM DRY DENSITY DETERMINED IN ACCORDANCE WITH ASTM D 698.

5) ADDITIONAL LATERAL LOAD DUE TO SEISMIC FORCES ON RETAINING WALLS CAN BE EVALUATED BASED ON A TRIANGULAR LATERAL EARTH PRESSURE DISTRIBUTION WITH A MAXIMUM PRESSURE OF 14H AT THE GROUND SURFACE AND 0 AT THE BASE OF THE WALL, WHERE H IS THE HEIGHT OF THE WALL. THE RESULTANT FORCE ACTS AT A POINT ABOVE THE BASE OF THE WALL EQUAL TO 60% OF THE WALL HEIGHT.

GRI

KENNEDY / JENKS CONSULTANTS
CORNET BAY MARINA

MSE WALL DRAINAGE DETAIL

AUG. 2013 JOB NO. W1115 FIG. 6

APPENDIX A

Field Explorations and Laboratory Testing

APPENDIX A

FIELD EXPLORATIONS AND LABORATORY TESTING

FIELD EXPLORATIONS

Subsurface materials and conditions at the site were investigated between June 20 and 21, 2013, with three borings, designated B-1 through B-3. The approximate locations of the borings are shown on Figure 2. The borings were advanced to depths of about 38 to 80 ft using a using a truck-mounted drill rig provided and operated by Cascade Drilling of Woodinville, Washington. The upper 20 ft of the borings were completed using hollow-stem techniques, and mud-rotary techniques were used below that depth. All drilling and sampling operations were observed by a geologist from GRI, who maintained a detailed log of the materials and conditions disclosed during the course of the work.

Disturbed and undisturbed soil samples were typically obtained at 2.5-ft intervals of depth in the upper 20 ft and at 5-ft intervals below this depth. Disturbed samples were obtained using a standard split-spoon sampler. At the time of sampling, the Standard Penetration Test was conducted. This test consists of driving a standard split-spoon sampler into the soil a distance of 18 in. using a 140-lb hammer dropped 30 in. The number of blows required to drive the sampler the last 12 in. is known as the standard penetration resistance, or SPT N-value. The N-values provide a measure of the relative density of granular soils, such as sand, and the relative consistency, or stiffness, of cohesive soils, such as silt. The split-spoon samples were carefully examined in the field and representative portions were saved in airtight jars. All samples were returned to our laboratory for further examination and physical testing.

Relatively undisturbed 3.0-in.-O.D. Shelby tube samples were obtained by pushing the tubes into undisturbed soil using the hydraulic ram on the drill rig. The soils exposed in the ends of the Shelby tubes were examined and classified in the field. The ends of the tubes were sealed with rubber caps and returned to our laboratory for further examination and physical testing.

Logs of the borings are provided on Figures 1A through 3A. 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 numbers and types of samples are indicated. Farther to the right, N-values are shown graphically, along with natural moisture content values, percent passing the No. 200 sieve, Torvane shear strength values, and Atterberg limits. The terms used to describe the materials encountered in the borings are defined in Table 1A.

LABORATORY TESTING

General

All samples obtained from the field were returned to our laboratory for examination and testing. The physical characteristics were noted, and the field classifications were modified where necessary. The soil samples were field-screened for the presence of organic vapors using a photo-ionization detector (PID). Additional laboratory tests included determinations of natural moisture content, Atterberg limits, Torvane shear strength, undisturbed unit weight, and grain size.

Natural Moisture Content

Natural moisture content determinations were made in conformance with ASTM D 2216. The results are provided on Figures 1A through 3A.

Atterberg Limits

Atterberg limits determinations were performed on samples of the clay obtained from each boring. The tests were performed in substantial conformance with ASTM D 4318. The test data were used for soil classification purposes and as indicators of engineering properties of the silt and clay soils at the site. The results of the Atterberg limit determinations are shown on Figures 1A through 3A and the Plasticity Chart, Figure 4A.

Grain Size Analysis

Dry Sieve. Dry sieve analyses were performed for selected soil samples to evaluate grain size distribution and assist in material classification and permeability estimates. The testing was completed in substantial conformance with ASTM D 6913-04. Test results are show on the grain size distribution curves, Figures 5A through 7A.

Washed Sieve. Washed sieve analyses were performed on representative soil samples to assist in their classification. The test is performed by taking a sample of known dry weight and washing it over a No. 200 sieve. The material retained on the sieve is oven-dried and weighed, and the percentage of material passing the No. 200 sieve is calculated. The test results are provided on Figures 1A through 3A.

Undisturbed Unit Weight

The unit weight, or density, of six undisturbed soil samples was determined in the laboratory in substantial conformance with ASTM D 2937. The unit weight determinations are summarized in the following table.

SUMMARY OF UNIT WEIGHT DETERMINATIONS

Boring	Sample	Depth, ft	Dry Unit Weight, pcf	Natural Moisture Content, %	Soil Type
B-1	S-4	10.5	110	20	Silty SAND; some clay, scattered organics and gravel (FILL)
	S-11	35	89	35	CLAY; trace to some silt, trace sand
B-2	S-3	8	102	26	Sandy SILT; trace to some clay, scattered gravel (FILL)
	S-7	18	105	26	Sandy CLAY; some silt
B-3	S-6	16.5	100	29	Silty CLAY; some sand, scattered gravel
	S-10	31	109	23	Silty CLAY; some sand, scattered gravel

Torvane Shear Strength

The approximate undrained shear strength of relatively undisturbed soil samples was determined using a Torvane shear device. The Torvane is a hand-held apparatus with vanes that are inserted into the soil. The torque required to fail the soil in shear around the vanes as the instrument is rotated is measured using a calibrated spring. The results of the Torvane shear tests are shown on Figures 1A through 3A.

Table 1A

GUIDELINES FOR CLASSIFICATION OF SOIL

Description of Relative Density for Granular Soil

Relative Density	Standard Penetration Resistance (N-values) blows per foot
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 foot	Torvane Undrained Shear Strength, tsf
very soft	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

Sandy silt materials, which exhibit general properties of granular soils are given relative density description.

Grain-Size Classification

Boulders
12 - 36 in.

Cobbles
3 - 12 in.

Gravel
1/4 - 3/4 in. (fine)
3/4 - 3 in. (coarse)

Sand
No. 200 - No. 40 sieve (fine)
No. 40 - No. 10 sieve (medium)
No. 10 - No. 4 sieve (coarse)

Silt/Clay - pass No. 200 sieve

Modifier for Subclassification

Adjective	Percentage of Other Material In Total Sample
clean	0 - 2
trace	2 - 10
some	10 - 30
sandy, silty, clayey, etc.	30 - 50

CLASSIFICATION OF MATERIAL

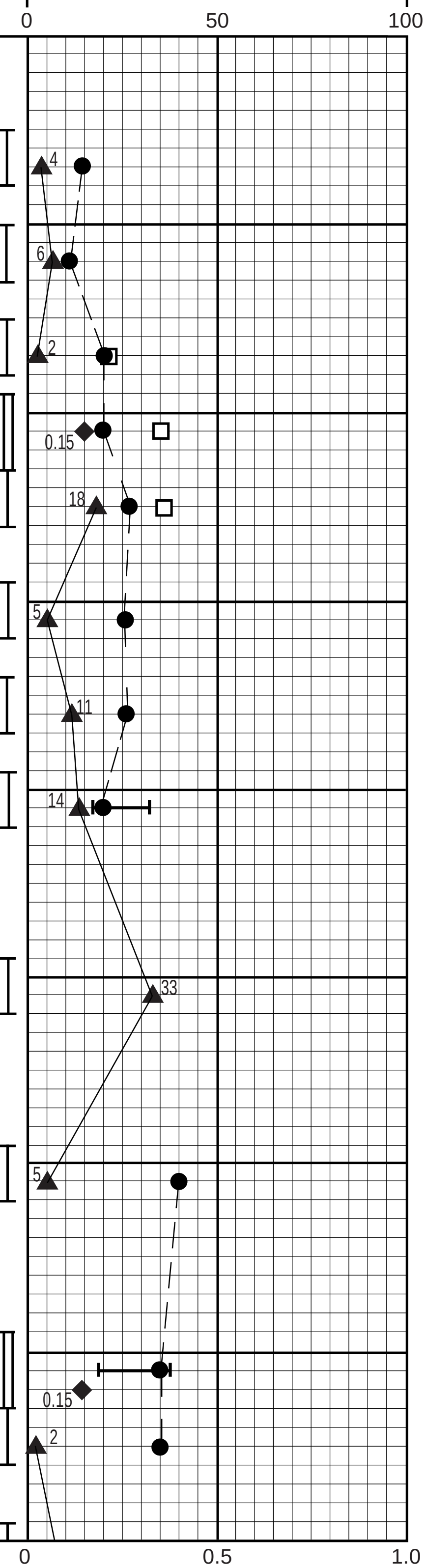
SURFACE ELEVATION 14 ft (±)

DEPTH, FT	CLASSIFICATION OF MATERIAL	DEPTH, FT	SAMPLES
0	FILL: Medium stiff, gray, sandy SILT; fine-grained sand, trace clay, scattered gravel		S-1
5	---------some sand, scattered organics at 5 ft		S-2
	FILL: Very loose, gray SAND; fine grained, some silt, clay, and gravel, scattered shells and organics	7.5	S-3
10	---------silt with some clay below 10.5 ft		S-4
	Loose to medium dense, gray, silty SAND; fine grained, some clay, trace to some gravel and shells	11.5	S-5
15			S-6
	Stiff, gray, silty CLAY; trace fine-grained sand	17.0	S-7
20	---------sandy at 20 ft; trace to some silt, trace fine-grained sand below 20 ft		S-8
25	---------hard at 25 ft		S-9 *
30	---------medium stiff below 30 ft		S-10
35	---------soft below 35 ft		S-11
			S-12
40	Medium stiff to stiff, gray, sandy SILT; fine- to coarse-grained sand, trace to some clay	39.5	S-13

GROUNDWATER

STD PENETRATION RESISTANCE
(140-LB WEIGHT, 30-IN. DROP)

▲ BLOWS PER FOOT
● MOISTURE CONTENT, %



2-IN.-OD SPLIT-SPOON SAMPLER
3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
NX CORE RUN
SLOTTED PVC PIPE
Water Level (date)

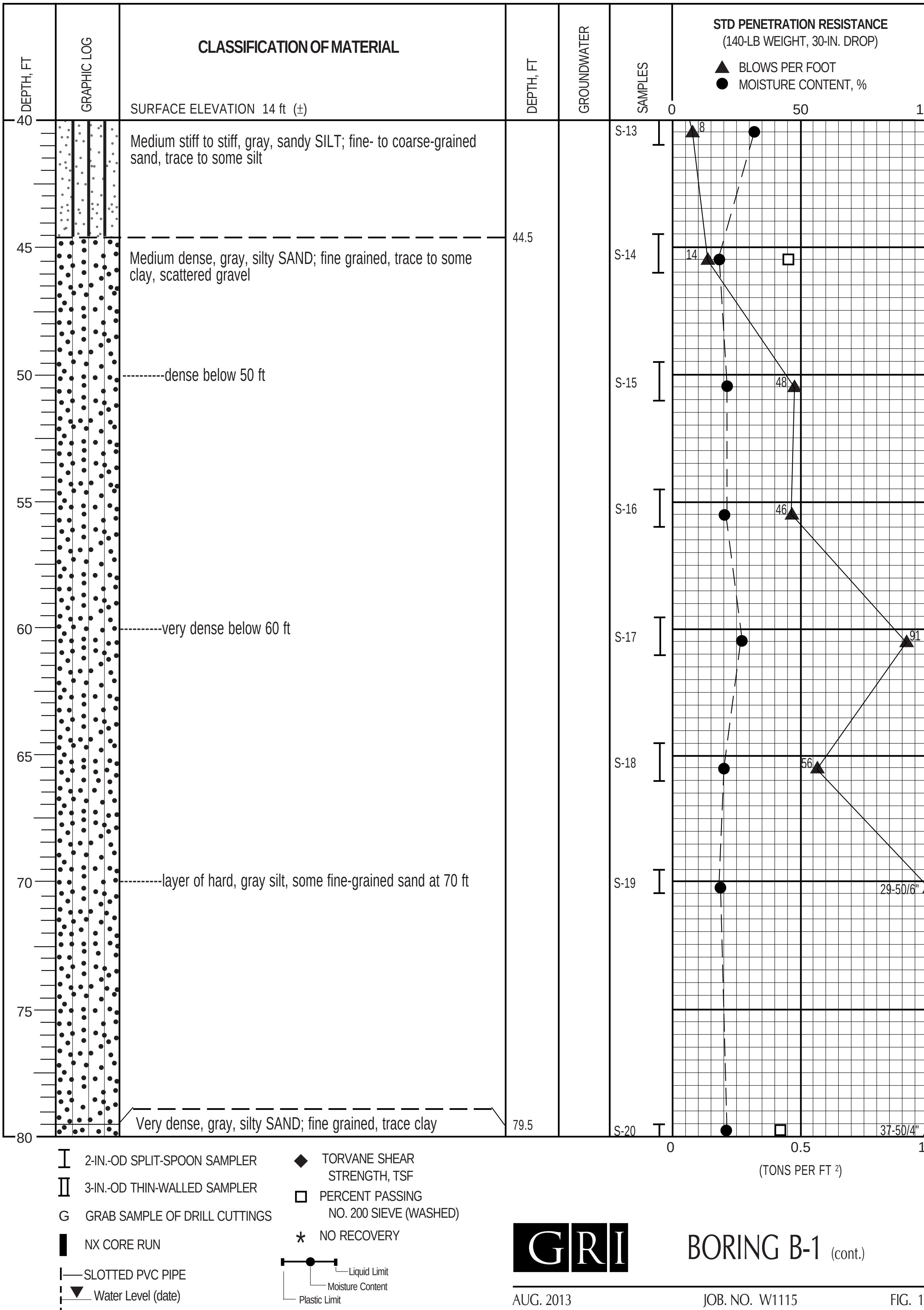
◆ TORVANE SHEAR STRENGTH, TSF
□ PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY

Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-1

AUG. 2013 JOB. NO. W1115 FIG. 1A



GRI

BORING B-1 (cont.)

AUG. 2013 JOB. NO. W1115 FIG. 1A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	DEPTH, FT	GROUNDWATER	SAMPLES	STD PENETRATION RESISTANCE (140-LB WEIGHT, 30-IN. DROP) ▲ BLOWS PER FOOT ● MOISTURE CONTENT, %
		SURFACE ELEVATION 14 ft (±)				0 50 100
80		Very dense, gray, silty SAND; fine grained, trace clay	80.3		I	
		(6/20/2013)				
85						
						0 0.5 1.0 (TONS PER FT 2)

I 2-IN.-OD SPLIT-SPOON SAMPLER

II 3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

▌ NX CORE RUN

|—SLOTTED PVC PIPE

▼ Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF

□ PERCENT PASSING NO. 200 SIEVE (WASHED)

* NO RECOVERY

Liquid Limit

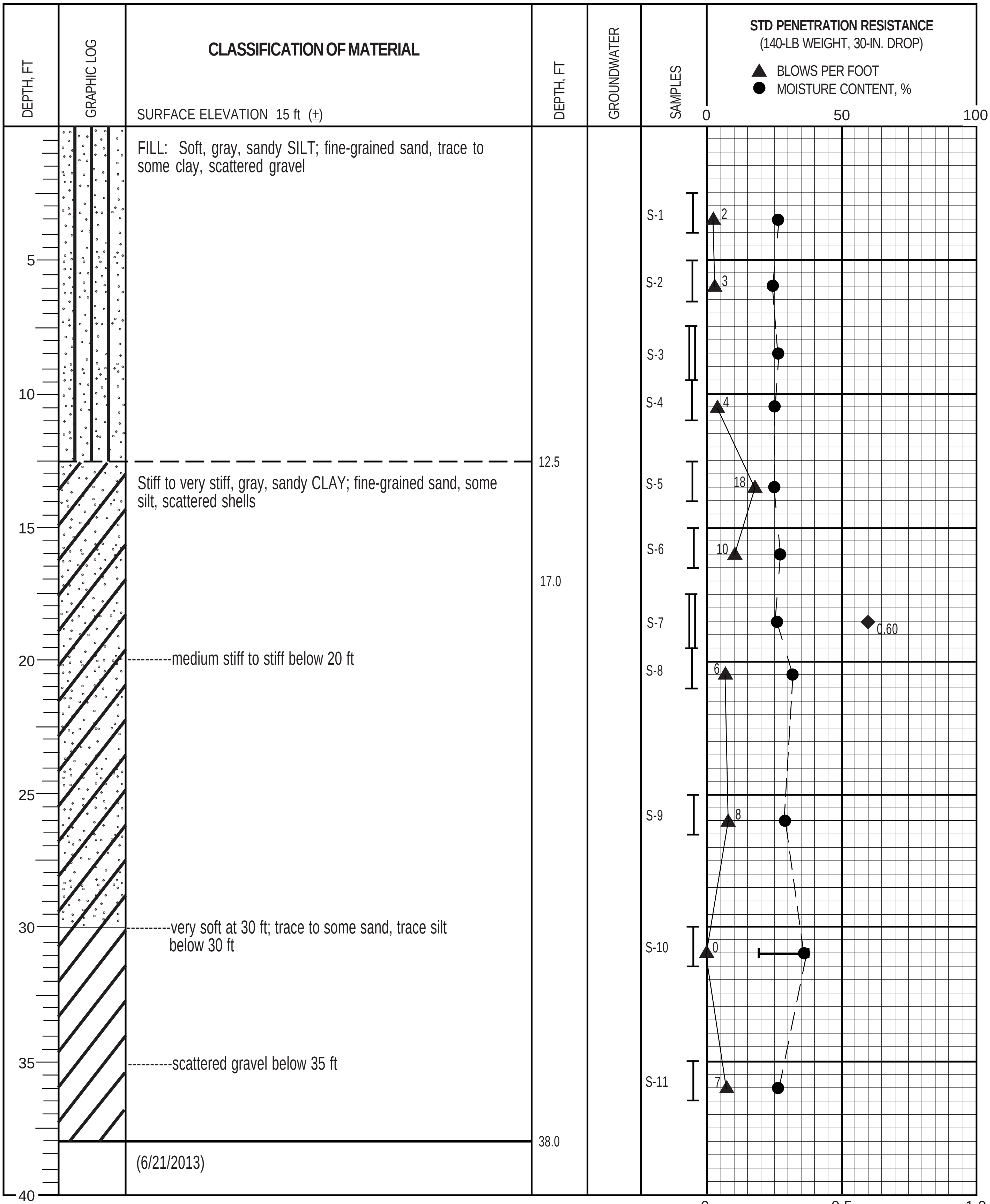
Moisture Content

Plastic Limit

GRI

BORING B-1 (cont.)

AUG. 2013 JOB. NO. W1115 FIG. 1A



2-IN.-OD SPLIT-SPOON SAMPLER

3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

NX CORE RUN

SLOTTED PVC PIPE

Water Level (date)

TORVANE SHEAR STRENGTH, TSF

PERCENT PASSING NO. 200 SIEVE (WASHED)

* NO RECOVERY

Liquid Limit

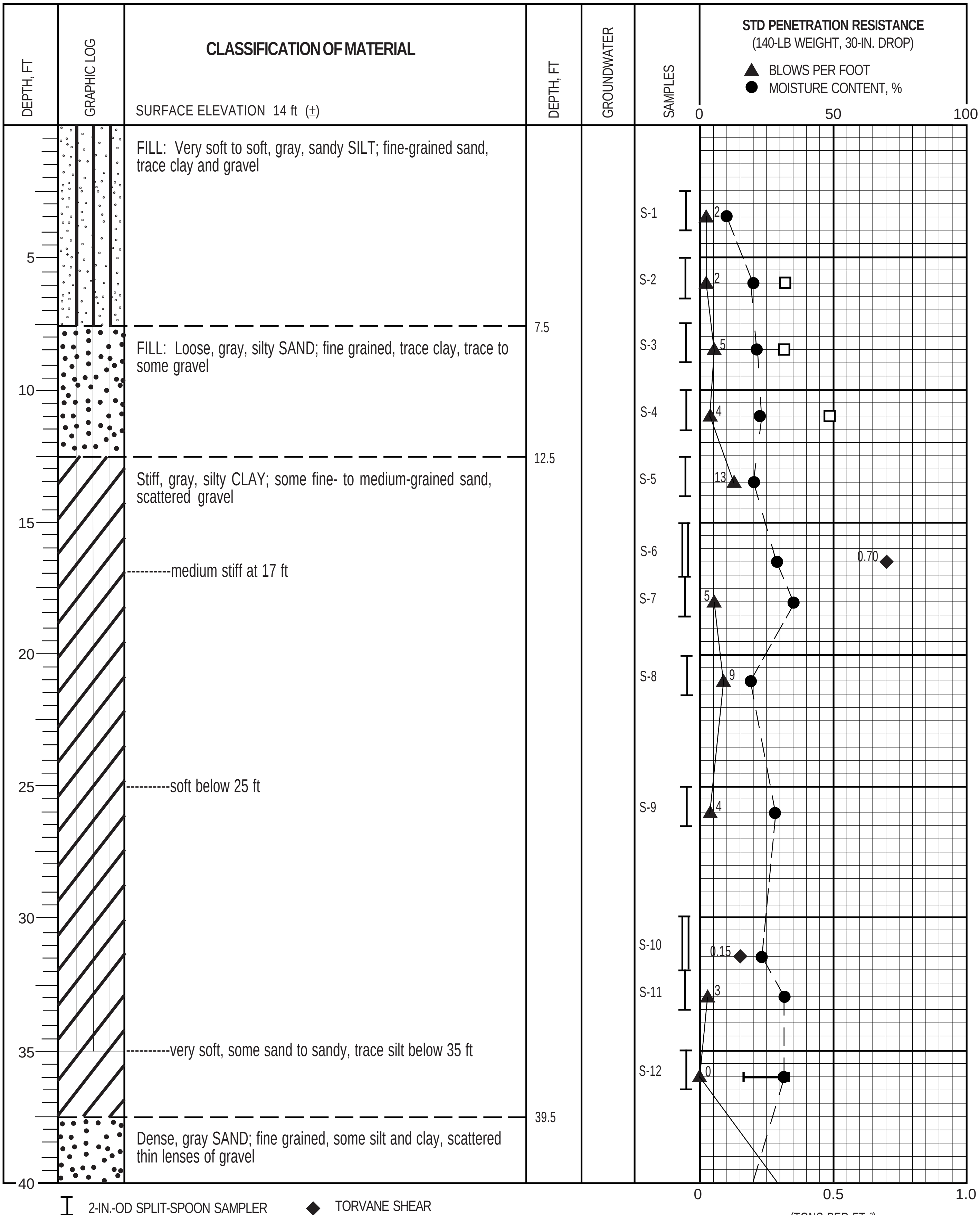
Moisture Content

Plastic Limit

GRI

BORING B-2

AUG. 2013 | JOB. NO. W1115 | FIG. 2A



- 2-IN.-OD SPLIT-SPOON SAMPLER
- 3-IN.-OD THIN-WALLED SAMPLER
- G GRAB SAMPLE OF DRILL CUTTINGS
- NX CORE RUN
- SLOTTED PVC PIPE
- Water Level (date)
- TORVANE SHEAR STRENGTH, TSF
- PERCENT PASSING NO. 200 SIEVE (WASHED)
- * NO RECOVERY
- Liquid Limit / Moisture Content / Plastic Limit

GRI

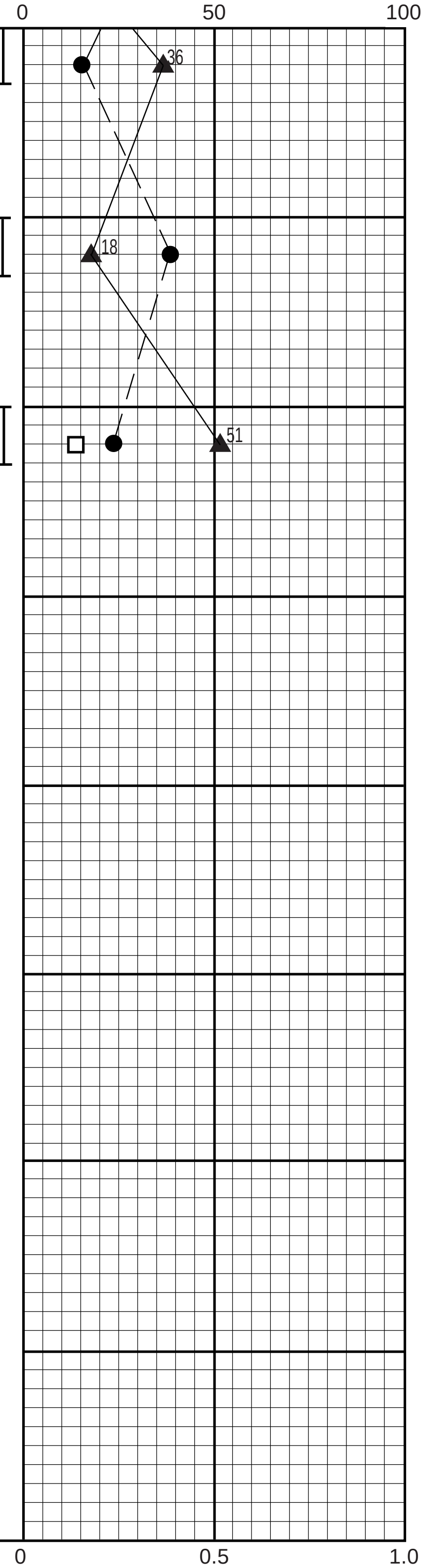
BORING B-3

AUG. 2013 | JOB. NO. W1115 | FIG. 3A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	DEPTH, FT	GROUNDWATER	SAMPLES
		SURFACE ELEVATION 14 ft (±)			
40		Dense, gray SAND; fine-grained, some silt and clay, scattered gravel			S-13
45		Very stiff, gray SILT; some clay and fine-grained sand	45.0		S-14
50		Very dense, gray SAND; fine grained, some silt and clay	50.0		S-15
		(6/21/2013)	51.5		
55					

STD PENETRATION RESISTANCE
(140-LB WEIGHT, 30-IN. DROP)

▲ BLOWS PER FOOT
● MOISTURE CONTENT, %



I 2-IN.-OD SPLIT-SPOON SAMPLER
II 3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
▌ NX CORE RUN
|— SLOTTED PVC PIPE
▼ Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF
□ PERCENT PASSING NO. 200 SIEVE (WASHED)
* NO RECOVERY

Liquid Limit
Moisture Content
Plastic Limit

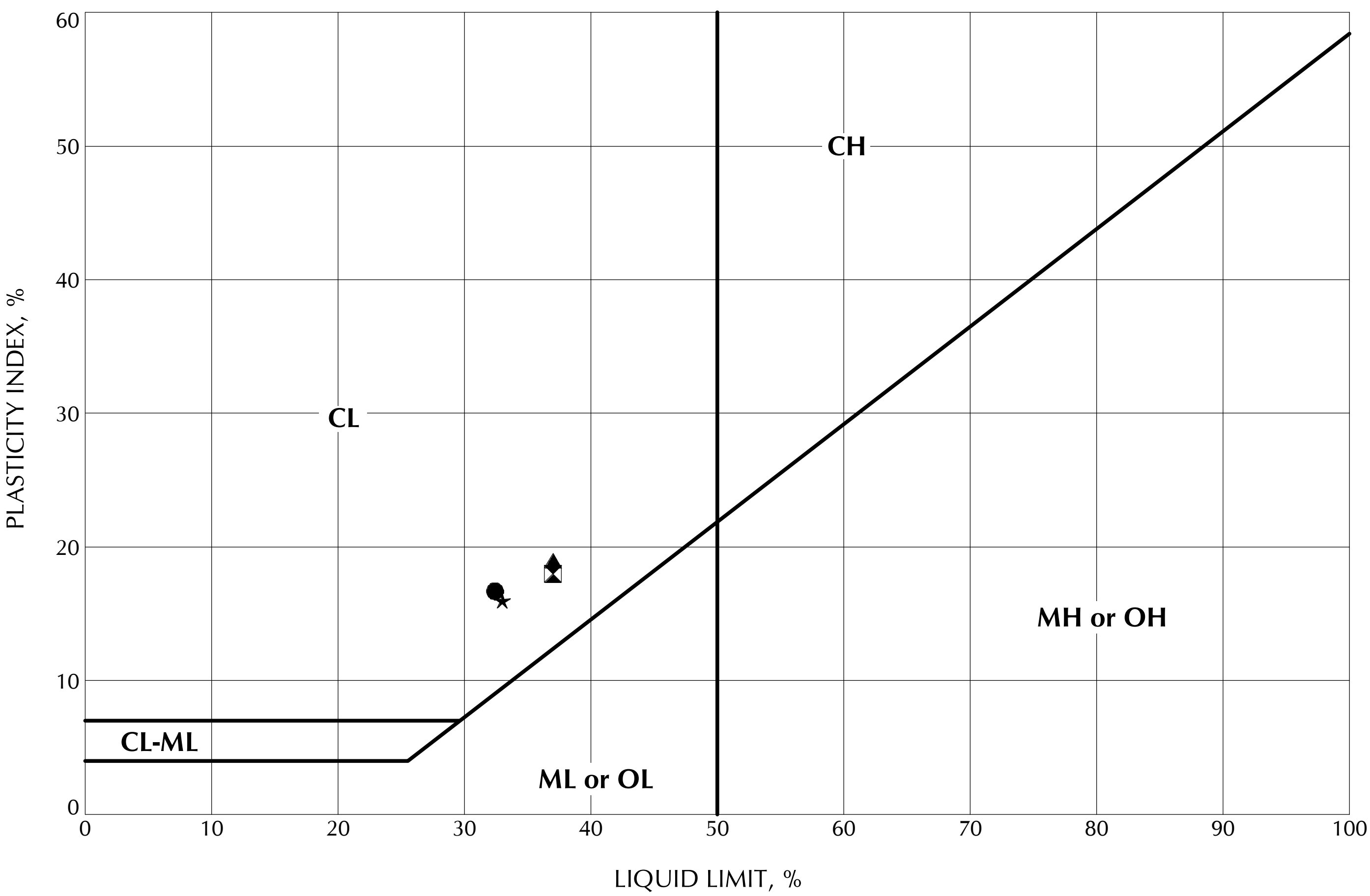
GRI

BORING B-3 (cont.)

AUG. 2013 JOB. NO. W1115 FIG. 3A

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-8	20.5	Sandy CLAY; fine-grained sand	32	16	16	20
⧗	B-1	S-11	35.5	CLAY; some silt, trace sand	37	19	18	35
▲	B-2	S-10	31.0	CLAY; trace silt and sand	37	18	19	36
★	B-3	S-12	36.0	CLAY; some sand, trace silt	33	17	16	31

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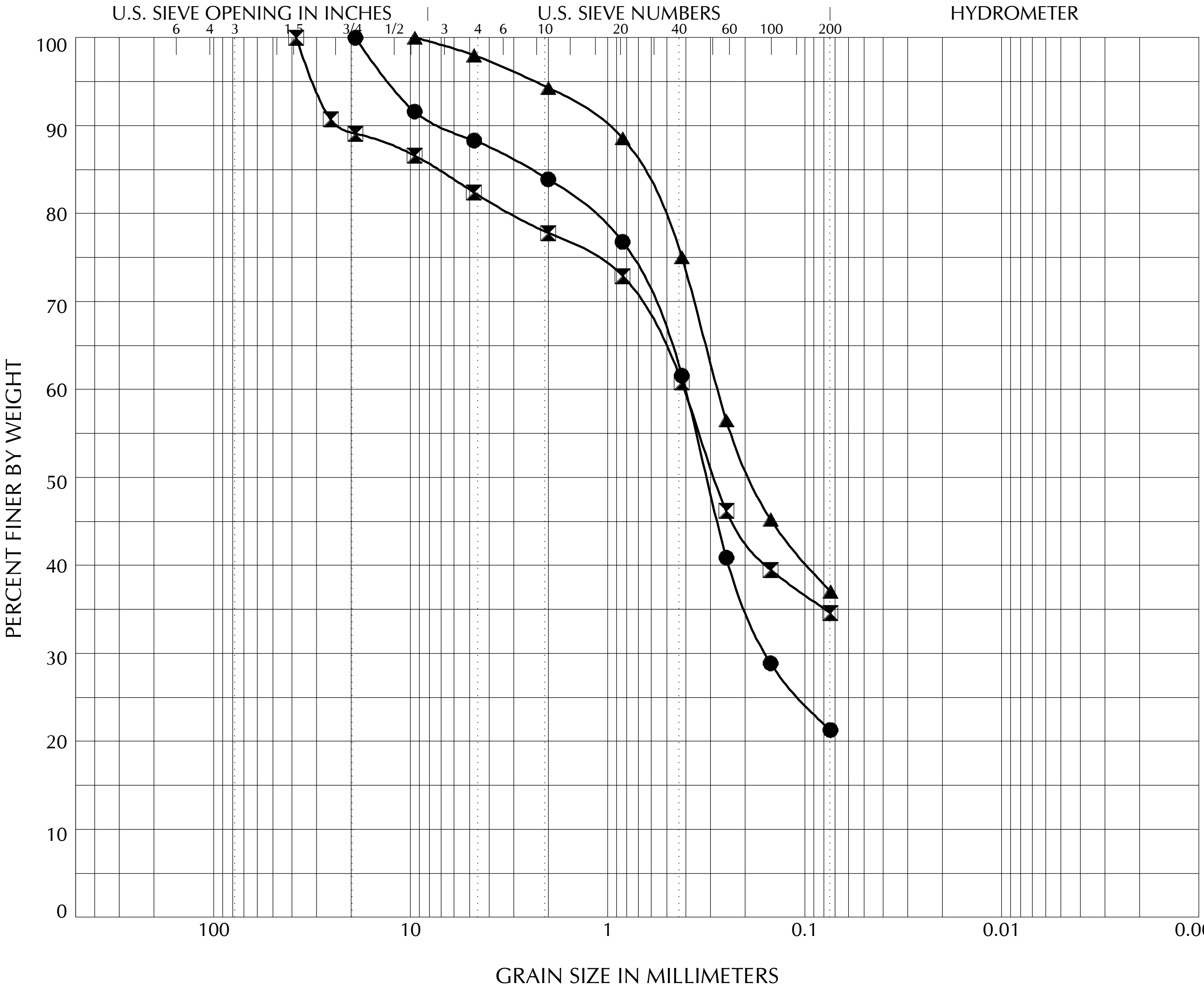
PLASTICITY CHART

AUG. 2013 PROJECT NO. W1115 FIG. 4A

ATTERBERG-PLASTICITY 4 PER PAGE W1115-LAB.GPJ GRI-LAB.GDT 8/27/13

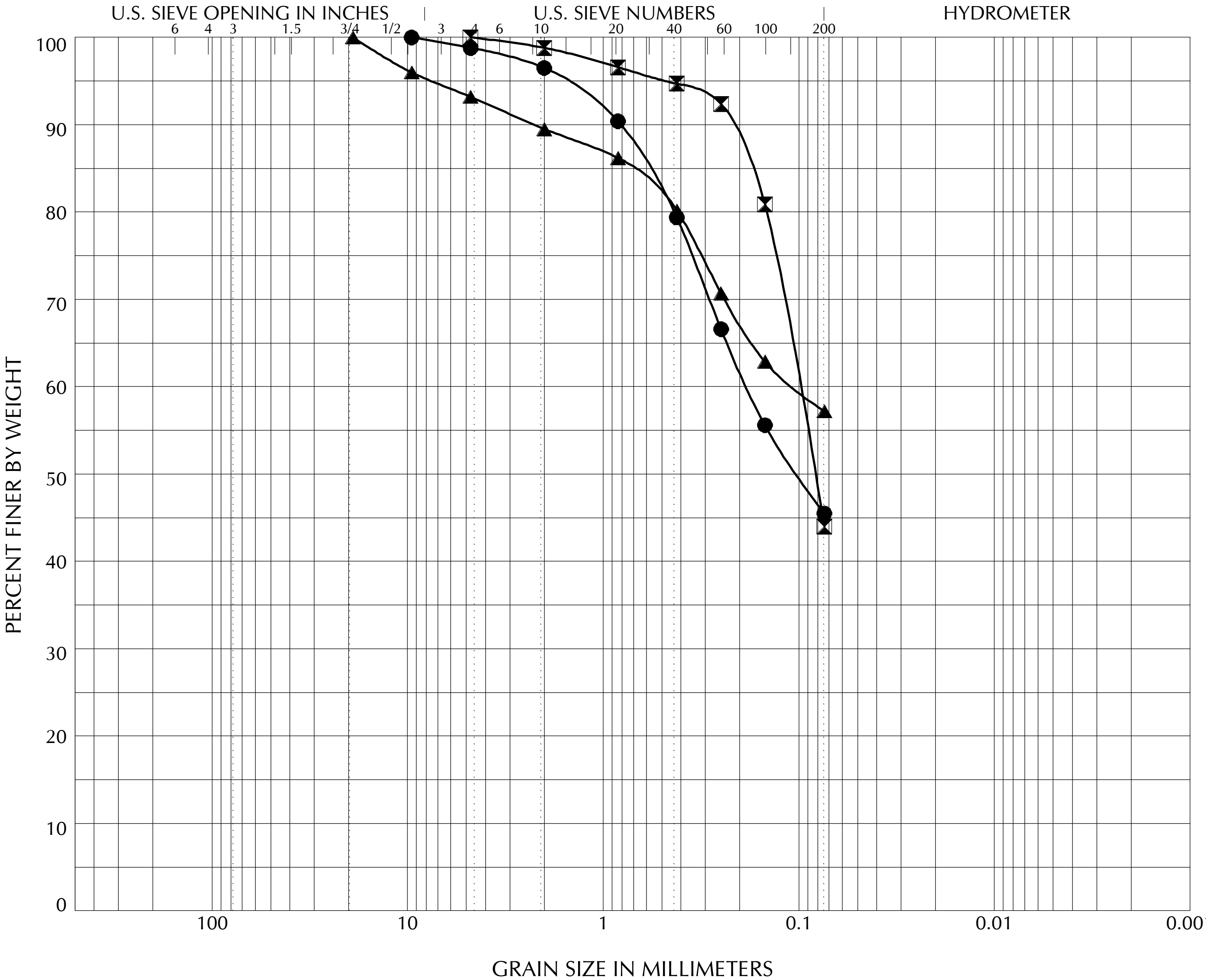


<table>
<tr><td rowspan="2">COBBLES</td><td colspan="2">GRAVEL</td><td colspan="3">SAND</td><td rowspan="2">SILT OR CLAY</td></tr>
<tr><td>Coarse</td><td>Fine</td><td>Coarse</td><td>Medium</td><td>Fine</td></tr>
</table>

	Location	Sample	Depth, ft	Classification	Gravel, %	Sand, %	Fines, %
●	B-1	S-3	7.5	FILL: SAND; some silt, clay, and gravel	11.7	66.8	21.4
⧗	B-1	S-4	10.5	FILL: Silty SAND; some gravel	17.6	47.7	34.7
▲	B-1	S-5	11.5	Silty SAND; trace gravel	2.0	60.8	37.2

GRI

GRAIN SIZE DISTRIBUTION



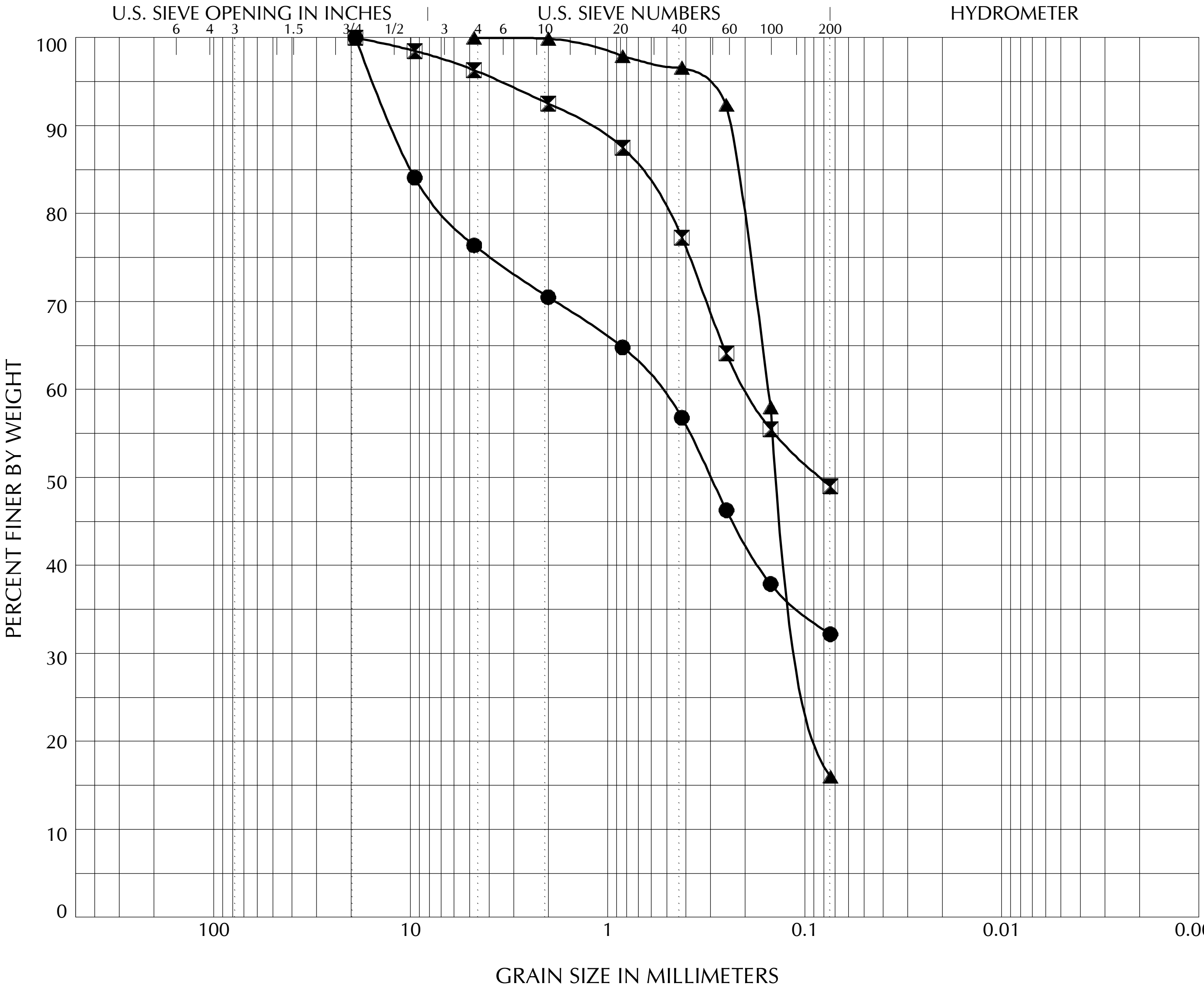
COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

	Location	Sample	Depth, ft	Classification	Gravel, %	Sand, %	Fines, %
●	B-1	S-14	44.5	Silty SAND; scattered gravel	1.2	53.1	45.7
⧗	B-1	S-20	79.5	Silty SAND; trace clay	0.0	55.3	44.7
▲	B-3	S-2	5.0	FILL: Sandy SILT; trace gravel and clay	6.8	35.9	57.3

GRI

GRAIN SIZE DISTRIBUTION

AUG. 2013 PROJECT NO. W1115 FIG. 6A



COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

	Location	Sample	Depth, ft	Classification	Gravel, %	Sand, %	Fines, %
●	B-3	S-3	7.5	FILL: Silty SAND; some gravel, trace clay	23.6	44.1	32.3
⧗	B-3	S-4	10.0	FILL: Sandy SILT; trace gravel and clay	3.7	47.2	49.1
▲	B-3	S-15	50.0	SAND; some silt and clay	0.0	83.2	16.8

GRI

GRAIN SIZE DISTRIBUTION

AUG. 2013 PROJECT NO. W1115 FIG. 7A

GRAIN SIZE GRI-THREE PER PAGE W1115-LAB.GPJ GRI-LAB.GDT 8/20/13