Geotechnical Engineering Investigation Services

VA Medical Center

Renovate Basement Clinic Additions

Wilkes-Barre, Pennsylvania

GAI Project Number: F121805.00

February 19, 2013

Prepared for: Paradigm Engineers and Constructors
PO Box 436223
Louisville, KY 40253

Prepared by: GAI Consultants, Inc.
Philadelphia Office
1055 Westlakes Drive, Suite 200
Berwyn, Pennsylvania 19312

gai consultants
transforming ideas into reality®

February 19, 2013

F121805.00

Paradigm Engineers and Constructors
PO Box 436223
Louisville, KY 40253

Attention: Mr. Blaine Van Gansbeke, P.E., PMP, LEED AP, MBA

VA Medical Center – Basement Clinic Additions
Wilkes-Barre, Pennsylvania
Geotechnical Engineering Investigation Services

Dear Mr. Van Gansbeke:

In accordance with your request, GAI Consultants, Inc. (GAI) performed a Geotechnical Engineering Investigation in conjunction with the two proposed site entrance canopy features that will be constructed during renovations at the VA Medical Center in Wilkes-Barre, Pennsylvania. This geotechnical engineering investigation and analysis was performed in accordance with our Proposal dated January 8, 2013.

The purpose of this study was to understand the site subsurface conditions and to provide recommendations from a soils engineering viewpoint for the design and construction of the proposed canopy structure foundations. The information obtained, together with our interpretation of the findings, is presented in this report.

We appreciate the opportunity to perform this geotechnical investigation. If you have any questions or require additional information, please contact Bob Sabanas, P.E. at 610.640.7456 (ext. 2804) r.sabanas@gaiconsultants.com, or Michael Kwiatkowski, P.E. at 610.640.7456 (ext. 2846) j.kwiatkowski@gaiconsultants.com.

Sincerely,
GAI Consultants, Inc.

Michael J. Kwiatkowski, P.E.
Senior Project Engineer

Robert M. Sabanas, P.E.
Senior Engineering Manager

MJK/RMS/srw

cc: Sydney Goetz, Paradigm Engineers and Constructors

Table of Contents

1.0 Introduction

GAI Consultants, Inc. (GAI) has prepared this report, which presents the conclusions and recommendations of our geotechnical engineering analysis conducted for the two proposed exterior canopy features along the western facility entrance at the VA Medical Center in Wilkes-Barre Township, Luzerne County, Pennsylvania. This engineering analysis was performed in accordance with our Geotechnical Consulting Services proposal dated January 8, 2013.

The scope of our services included the completion of four (4) test borings, performance of laboratory testing of representative soil samples, geotechnical analyses of the findings, and the preparation of a written engineering report. This report documents the work performed, describes the site conditions, and presents pertinent geotechnical engineering conclusions and recommendations.

2.0 Project Description

VA Medical Center is proposing a series of renovations to their existing facility including two new exterior canopy features for the western facility entrance, which is the focus of this study. The VA Medical Center is located on the east side of Mundy Street, between Bear Creek Boulevard and SR 309 in Wilkes-Barre Township, Pennsylvania. Refer to the Site Location Map (Figure 1) in Appendix A for the location of the project.

The proposed canopy features will provide expanded cover for the proposed facility entrance and associated walkways. Based on a review of the plans provided, we understand that the proposed walk ways and new canopy features will be located in current asphalt paved areas of the existing facility entrance. Some of the asphalt and existing walkways will be reconfigured as part of the renovation work.

The structure will consist of a 15 to 18-foot wide, steel framed canopy structure, and will cantilever from the base support feature. The maximum anticipated design loads (assumed to act at the pile head), as provided by Steve Leonard Consulting Engineer, PLLC, are as follows:

- Vertical Compression = 15 kips
- Vertical Uplift = 5 kips
- Shear = 7.5 kips
- Overturning = 80 ft-kips

We assume that the post-construction vertical and lateral deflection tolerances are 1 inch and 0.5 inches, respectively.

In the event the nature, design or location of the proposed construction changes, or our assumptions and understandings of the proposed construction are incorrect, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed, and the conclusions are modified and confirmed in writing by GAI.

3.0 Subsurface Investigation

A Standard Penetration Test (SPT) boring investigation was performed on February 2, 2013. The investigation consisted of four (4) SPT test borings drilled by Heller Drilling Company, Inc. of Wilkes-Barre, PA using a Diedrich D50 truck mounted drilling rig and hollow-stem auger drilling methods, at locations selected in the field by a representative of GAI. The soil borings, designated as B-1 through B-4, were located in grass or landscaped areas along the western entrance of the facility and were drilled to auger and sampling spoon refusal at depths ranging from 5 ft to 13 ft below existing grade. Prior to initiating each test boring, the location was cleared with respect to underground utilities by our private utility locator, Master Locators. In addition, GAI notified the Pennsylvania One-Call utility locating service 72-hours prior to mobilizing to have the utility companies check for and mark public underground lines. GAI provided full-time technical supervision during the field exploration.

Soil samples were recovered from the test borings via a two-inch O.D. split-spoon sampler driven by a 140-pound hammer, free falling 30 inches (ASTM D 1586). The number of hammer blows required to advance the 24-inch spoon in 6-inch increments (four increments in all) were recorded. The number of blows required to penetrate the middle two increments (6 to 18 inches) is known as the Standard Penetration Resistance (N). Soil samples were typically obtained continuously in the upper 10 ft and at 5 ft intervals thereafter. Recovered soil samples were visually classified and logged using the *Unified Soil Classification System.*

The approximate location of the test borings, along with other pertinent site information, is shown on the Test Boring Location Plan (Figure 2) in Appendix A, which is based on drawings provided by Paradigm Engineers and Constructors. The test boring logs are presented in Appendix B, along with a Glossary of Terms and Definitions.

4.0 Laboratory Testing

Recovered soil samples were taken to GAI's materials laboratory located in King of Prussia, PA, for examination and testing. The field classifications were confirmed or modified as necessary by a Geotechnical Engineer or Geologist. Laboratory tests were performed on selected samples obtained from the test borings to characterize the index and classification properties of the subsurface soils. The tests performed on representative soil samples included visual classification, water content (ASTM D 2216) and mechanical sieve analysis (ASTM D 422). Graphic presentations of the laboratory test results are shown in Appendix C.

5.0 Site and Subsurface Conditions

5.1 Published Geologic Data

Published geologic data indicates that the site is underlain by the Llewellyn Formation (lPl), which consists of interbedded sandstone, siltstone, and conglomerate. This formation is moderately well developed with joints having a blocky pattern that are moderately developed, moderately abundant and moderately spaced. The Llewellyn Formation is slightly to moderately weathered from a shallow to moderate depth, and has good surface drainage. The foundation stability in this formation is good. This description is consistent with the soil samples encountered during the field exploration.

Site geology was obtained from "Engineering Characteristics of the Rock of Pennsylvania," Environmental Geology Report 1, Department of Environmental Resources, Bureau of Topographic and Geologic Survey, 1982.

5.2 Soil Encountered

The site soils are in general agreement with the published geologic data. Based on the borings, the subsurface soil profile generally consists of topsoil, underlain by coarse-grained decomposed rock, underlain by completely to partially weathered bedrock grading to competent bedrock. A detailed description of the soils encountered is shown on the boring logs. A brief general description is given in the following sections.

5.2.1 Topsoil

Topsoil consisting of a dark brown sand and silt mixture was encountered in each of the test borings. The thickness of the topsoil layer was about 6 inches at each test boring location. However, the thickness will likely vary within the footprint of the proposed canopy feature (i.e. no topsoil anticipated in current paved areas).

5.2.2 Decomposed Rock

Decomposed rock was encountered in each of the test borings beneath the topsoil layer ranging from about 3 feet to 5 feet below the ground surface. Decomposed rock represents the partial weathering of the parent bedrock in that it retains some of its relic structure. The decomposed rock at the project site consists of a grey to dark grey sand and gravel mixture, with sandstone and siltstone rock fragments. Standard penetration test (SPT) values for the decomposed rock range from 7 blow per foot (bpf) to about 50 blows per 6 inches, with an average of about 35 bpf. Increased resistance with depth was observed in each of the test borings.

The upper 1 to 2 feet of the decomposed rock layer may have been previously disturbed (i.e. during previous site construction activities) and therefore may be representative of fill material. However, clear evidence or visual indicators of fill material such as foreign materials (foreign soils, non-uniform matrix, bricks, concrete, asphalt, etc.) were not observed in this zone. As a result, a fill layer was not readily distinguishable from the decomposed rock layer for the purposes of this subsurface soil description.

5.2.3 Weathered Rock

Weathered rock consisting of gravel (rock fragments) and sand was encountered in each of the test borings beneath the decomposed rock layer. The weathered rock is similar to the decomposed rock layer in that it retains some of its relic rock structure. However, for purposes of this report, the weathered rock zone results in sampler refusal (i.e. greater than 50 blows per 6 inches), but can be penetrated with hollow stem augers.

The weathered rock layer at the site is somewhat variable based on the observed auger refusal depths, which ranged from 5 feet at B-2 to 13 feet at B-4. As a result, some variability in the thickness of the weathered rock zone should be expected. Alternating zones of more and less weathered rock may be present, as observed in test boring B-4, where auger refusal appeared imminent at about 8 feet below the ground surface, consistent with the refusal depths at B-1 through B-3, but was eventually advanced to a depth of about 13 feet with some difficultly.

5.2.4 Intact Rock

Auger refusal, suggesting the presence of intact rock, was encountered in each of the test borings at depths ranging from 5 to 13 feet, and averaging 8 feet below the ground surface. The upper several feet of the intact rock layer is likely highly fractured based on the relatively thick and variable weathered rock zone. No rock coring was completed during this field study.

5.3 Groundwater Conditions

Groundwater was not encountered at any borings at the time of the drilling. Additionally, the soil samples were observed to generally be in a dry to slightly moist condition.

It should be noted that this groundwater information represents the conditions encountered at the time of the drilling operations. Groundwater levels generally can fluctuate due to changes in precipitation, infiltration, surface run-off, and other hydrogeological factors. Therefore, the groundwater level present at the time of construction may vary from that detected at the time of the drilling operations.

It should also be noted that shallow perched groundwater may be encountered during construction, especially if the work commences after a wet weather period. Dewatering of perched water or surface runoff water encountered during construction can be performed using sump pumps.

6.0 DISCUSSION & RECOMMENDATIONS

6.1 Canopy Foundations

We have evaluated the existing subsurface soil conditions to determine the most appropriate foundation alternatives, and performed analyses to develop appropriate corresponding engineering design parameters for the support of the proposed canopy features. Based on the subsurface conditions at the site and the anticipated loads, we recommend that the proposed canopy be supported on drilled piers (caissons). Based on the structural requirements and our analyses, drilled piers are the most feasible option for resistance of the lateral and overturning moment loads compared to alternative foundation systems (i.e. oversized shallow foundations). Therefore, design recommendations for drilled piers are provided in this report.

Table 1 summarizes the maximum anticipated reaction loads acting on the foundation and the associated deflection tolerances, as determined by Steve Leonard Consulting Engineer, PLLC.

Table 1: Summary of Foundation Loads and Deflection Tolerances

Feature	Axial Load (kips)		Shear (kips)	Overturning Moment (ft-kips)	Deflection Tolerance (in.)	
	Comp.	Uplift			Vertical	Lateral
Canopy	15	5	7.5	80	1.0	0.5

The following sections summarize our design recommendations for the drilled pier foundation system. Recommendations including preliminary sizing is further discussed for drilled piers.

6.2 Drilled Piers (Caissons)

As discussed previously, due to the specified lateral and overturning moment loads, we recommend a drilled pier foundation system for this project. Considering the variability with respect to the intact rock depth (5 feet to 13 feet below the ground surface), we have provided design recommendations for both drilled piers bearing in soil (i.e. the weathered rock layer) and drilled piers bearing in the intact rock layer.

Our preliminary analyses indicate a 36 inch diameter drilled pier bearing within the weathered rock at a minimum overall depth of 12 feet below the existing ground surface elevation will adequately support the proposed canopy feature. If intact rock is encountered prior to 12 feet, the drilled piers must be extended into the rock a minimum of 3 feet (i.e. rock socket) for lateral stability. Refer to Section 6.3.2 for additional information with respect to encountering shallow rock.

The preliminary sizing of the drilled piers is for information only. The final design of the drilled piers should be confirmed by the design engineer using the soil parameters provided in this report. The following is a summary our recommendations for the drilled piers.

6.2.1 Axial Capacity

The drilled pier foundations should be dimensioned to provide sufficient vertical axial (compressive) support via end bearing and skin friction, or vertical uplift resistance via skin friction and dead weight. As previously discussed, the preliminary design of the drilled piers is not governed by axial capacity. It is governed by the lateral / overturning moment loads (see Section 6.2.2). As a result, the final drilled pier dimensioning should take this into consideration.

Based on the results of the test borings conducted at the site, we have estimated the soil and rock parameters listed in Table 2, which should be used for axial capacity design of the drilled pier foundations. A linear variation between depths may be assumed for values of skin friction. A minimum factor safety of 3 should be applied to the ultimate skin friction and end bearing values. This factor of safety may be reduced by 30 percent when designing for transient (wind or seismic) loads.

Table 2: Soil Parameters for Drilled Pier Foundation Design Bearing

Soil Layer (depth)	Effective Unit Weight (pcf)	Angle of Internal Friction	Modulus of Lateral Subgrade Reaction, k (pci) (4)	Depth (FT)	Ultimate Skin Friction (2)(3) (psf)	Ultimate End Bearing (ksf)
1 (0 ft. to 2 ft.)	110	28°	25	0 2	0 0	N/A N/A
2 (2 ft. to 5 ft.)	120	33°	90	2 5	400 1,070	N/A 12
3 (5 ft. to 10 ft.)	130	35°	225	5 10	1,070 2,070	24 24
4 (> 10 ft.)	140	38°	N/A	>10	3,000	75

SOIL DESCRIPTIONS:

Layer 1: Potential fill or disturbed decomposed rock layer. Conservatively neglect for vertical capacity analysis.
Layer 2: Decomposed Rock.
Layer 3: Weathered Rock (Extent of Weathered Rock layer varied from 5 to 13 feet deep).
Layer 4: Intact Rock; May be encountered as shallow as 5 feet deep.
(For intact rock lateral analysis, use Young's Modulus = 145,000 psi; k_{rm} = 0.0005).
(1) Table 2 is based on generalized soil profile for this site.
(2) Assume a linear variation between depths for ultimate skin friction. If the drilled pier if bearing within the intact rock, the ultimate skin friction values provided for layers 1 through 3 should be ignored.
(3) The ultimate skin friction values shown are for use in drilled pier design in compression. For uplift (tensile) resistance, assume 50% of the ultimate skin friction value in compression for use in design.
(4) K-value is to be used in conjunction with L-Pile software only.
N/A = Not applicable

The allowable vertical capacity of the preliminary drilled pier design (36-inch diameter pier bearing within the weathered rock or the intact rock layer) exceeds the maximum vertical reaction loads (15 kips (compression) or 5 kips (uplift)). Post-construction settlement of the drilled pier bearing in weathered rock or intact rock is expected to be less than 0.5 inches, which is less than the post-construction limit.

6.2.2 Lateral Deflection and Moment Capacity Analysis Using LPILE

Lateral deflection of the preliminary sized drilled pier was analyzed using LPILEv5.0 computer program using the parameters provided in Table 3. A minimum vertical reinforcement of 1% of the gross cross sectional area and 3 inches of concrete cover were chosen. In addition, the reinforcement was assumed to be equally spaced in a circular fashion with a nominal bar size of No. 9. It was assumed that the top of the drilled pier was free to rotate, and the ultimate bending moment capacity of the concrete pier was limited to a maximum of 0.3% strain. Other considerations in the analysis included: a concrete compressive strength and Young modulus of 3,000 psi and 3,000,000 psi, respectively, and a yield stress and Young's modulus for the steel reinforcement of 60 ksi and 29,000 ksi, respectively.

Using the design loads and the soil parameters summarized in Table 2, LPILEv5.0 was used to determine the required drilled pier dimensions to limit the drilled pier ground line deflection to a maximum of 0.5 inches. Based on the soil profile for this project, a 36-inch diameter drilled bearing in the weathered rock layer must be embedded a minimum of 12 feet to limit the deflection at the pile head to less than 0.5 inches. Accordingly, our preliminary design for the drilled pier consists of a minimum 36-inch diameter by 12-foot long drilled pier.

If sound coreable rock (as defined below) is encountered prior to the design depth (i.e. 12 feet), extend the shaft a minimum of 3 feet into the rock (i.e. rock socket). However, if sound coreable rock is encountered within 3 feet of the design length of the drilled pier, reduce the length of the rock socket such that the design length of the shaft is not exceeded. These recommendations are intended to provide adequate deflection resistance in the drilled pier in the event shallow rock is encountered. If sound coreable rock is encountered prior to a depth of 5 feet below the ground surface, GAI should be contacted for additional recommendations.

Sound coreable rock is defined as a stratum of geomaterial having an unconfined compressive strength equal to greater than 2,500 psi that cannot be drilled with conventional earth augers or underreaming tools, thus requiring the use of special rock augers. Excavation advancement criteria for sound rock is less than or equal to 4-minutes per foot using a rock auger with conical teeth.

6.2.3 Drilled Pier Construction

Temporary casing of the drilled piers will likely be necessary to prevent sloughing of the granular soils. Concrete placed near the surface should be in full contact with the natural-undisturbed soil to provide lateral stability for the full length of the drilled pier. The soil immediately surrounding the top of the drilled pier should be compacted following construction activities. We do not anticipate that groundwater will be encountered in the foundation excavations.

A qualified geotechnical engineering representative should be present at the time of drilled pier installation to confirm the bearing stratum. In addition, this will enable rapid response to potential unexpected conditions encountered in the drilled pier excavations.

6.3 Excavation and Backfill

If necessary, open cut excavations (i.e. for utility excavations or shallow foundations) can be used for this project provided that the temporary side slopes of the open cut excavation is not be steeper than 1.5H:1V. Additionally, any excavation close to existing structures may affect the existing foundation. Existing foundations may be considered not affected by the open cut excavation if a line projected downward from the bottom of the existing foundation at a slope of 1.5H:1V does not intersect the excavation slope. All excavations should be in compliance with "Excavating and Trenching Operations" manual (latest revision), issued by the US Department of Labor, OSHA 2226, and local requirements.

Backfilling of any excavation should be accomplished using controlled fill compacted to 95% of the maximum dry density, as determined by the Modified Proctor Test (ASTM D 1557).

6.4 Controlled Fill

Any required fill or backfill should be placed under controlled conditions. Controlled fill should consist of inorganic, readily compactable, predominantly well-graded granular soils. The on-site soils are considered suitable for use as controlled fill, assuming that the materials are readily compactable. It is recommended that fragments having a maximum dimension greater than 3 inches be excluded from the fill. The moisture content of the fill materials should be controlled to within 2% of the optimum moisture content by wetting, aeration or blending in order to facilitate compaction. If imported soils are required, we recommend material with a maximum of 15% fines (material passing the No. 200 sieve).

Controlled fill should be placed in loose horizontal lifts with a maximum thickness of 12 inches. It is recommended that controlled fill within the construction area be compacted to at least 95% of the maximum dry density as determined by the Modified Proctor Test (ASTM D 1577). In addition, it is recommended that all fills be visually stable under construction traffic, as observed by a geotechnical on-site representative of the geotechnical engineer.

6.5 Seismic Parameters

According to the Pennsylvania Edition of the 2009 International Building Code, the project site can be categorized as Site Class "C" for seismic design purposes. This classification is based on soil properties obtained from the borings to a maximum depth of 13 feet below the ground surface, and assumptions for the soil / rock thereafter to a depth of 100 feet.

7.0 LIMITATIONS

The conclusions and recommendations contained in this report are based upon the subsurface data provided and on details stated in this report. It is understood that the number of borings made are consistent with good engineering practice but actual conditions encountered may differ significantly from those projected in this report. Should conditions arise which differ from those described in this report, GAI should be notified immediately and provided with all information regarding differing subsurface conditions.

Our recommendations are based upon the assumption that the services of a qualified Geotechnical Engineer will be retained during construction for the observation of all critical earthwork operations and foundation installation. GAI cannot minimize, or provide recommended solutions for, any problems resulting from construction or differing soil conditions unless our services include full-time construction inspection to determine that the work performed is in compliance with GAI's recommendations, and to ensure the work is carried out in the owner's best interests.

Environmental considerations and contaminants, such as petroleum products, hazardous waste, radioactivity, irritants, pollutants, radon or other dangerous substances and conditions were not the subject of this study. Their presence and/or absence are not implied, inferred or suggested by this report or results of this study.

This report is intended for use with regard to the specific project discussed herein, and any changes in the design of the structure or location, however slight, should be brought to our attention so that we may determine how they may affect our conclusions. We are responsible for the conclusions and opinions contained in this report based on the data provided by Paradigm Engineers and Constructors relating only to the specific project and location discussed herein.

APPENDIX A
Figure 1: Site Location Map
Figure 2: Test Boring Location Plan

GRAPHIC IMAGE OBTAINED FROM GOOGLE EARTH

gai consultants Philadelphia Office 1055 Westlakes Drive, Suite 200 Berwyn, PA 19312 610-640-7456	SITE LOCATION MAP WILKES-BARRE VA MEDICAL CENTER NEW CANOPY FEATURES	DWN. J.M.R.	CHKD. M.K.	APPD. [illegible]	DATE 2-18-13
		SCALE: N.T.S.			TASK NO. __
	LUZERNE COUNTY WILKES-BARRE, PENNSYLVANIA	PROJECT NO./DASH NO. F121805.00			
		DRAWING NO. FIGURE 1			REV —

GRAPHIC IMAGE OBTAINED FROM GOOGLE EARTH

LEGEND:

B-1 ⌖ TEST BORING LOCATION

gai consultants Philadelphia Office 1055 Westlakes Drive, Suite 200 Berwyn, PA 19312 610-640-7456	TEST BORING LOCATION PLAN WILKES–BARRE VA MEDICAL CENTER NEW CANOPY FEATURES	DWN. J.M.R.	CHKD. M.K.	APPD. MM	DATE 2–18–13
		SCALE: N.T.S.			TASK NO. ___
	LUZERNE COUNTY WILKES–BARRE, PENNSYLVANIA	PROJECT NO./DASH NO. F121805.00 DRAWING NO. FIGURE 2			REV –

APPENDIX B
GAI Testing Boring Logs (4)
Key to Soil Symbols and Terms

TEST BORING LOG

gai consultants

PROJECT NAME: VA Medical Center - Renovate Basement Clinic Addition	BORING NO.: B-1
PROJECT NO.: F121805.00	SHEET 1 OF 1

TOWNSHIP: Wilkes Barre **COUNTY:** Luzerne **STATE:** Pennsylvania
DRILL RIG TYPE: Diedrich D50 Truck **DRILLING METHOD:** HSA **DATE(S) DRILLED:** 02/02/13-02/02/13
DRILLER/COMPANY: T. Turner / Heller Drilling, Inc. **FIELD ENGINEER:** E. Gosney **CHECKED BY:** M. Kwiatkowski
GROUND ELEVATION: ' **GROUNDWATER DEPTH:** Not Encountered **FT. TIME:** **HR.**

ELEVATION (FT)	DEPTH (FT)	SAMPLE TYPE	SAMPLE NO./ CORE RUN NO.	BLOW COUNTS PER 6 IN.	REC (%)	RQD (%)	WC (%)	LL/PL (%)	USCS	qu/Pene (TSF)	STRATA SYMBOL	MATERIAL DESCRIPTION
												Fill - Topsoil
		SPT	S-1	4-4-9-11	75							Sandy Gravel (gps), Blk. & Gry., Dry, Med. Dense
	2											
		SPT	S-2	12-19-20-50/2"	50							Silty Sand with Gravel (sm), Brn., Dry, Dense [Weathered Rock] [Auger Refusal @ 5.0']
	4											
												End of Boring @ 5.0'

GAI SELP2 BORING LOGS.GPJ 41320

SAMPLE LEGEND	GENERAL NOTES
SPT SAMPLE SHELBY TUBE ROCK CORE	

TEST BORING LOG

gai consultants

PROJECT NAME: VA Medical Center - Renovate Basement Clinic Addition	BORING NO.: B-2
PROJECT NO.: F121805.00	SHEET 1 OF 1

TOWNSHIP: Wilkes Barre **COUNTY:** Luzerne **STATE:** Pennsylvania

DRILL RIG TYPE: Diedrich D50 Truck **DRILLING METHOD:** HSA **DATE(S) DRILLED:** 02/02/13-02/02/13

DRILLER/COMPANY: T. Turner / Heller Drilling, Inc. **FIELD ENGINEER:** E. Gosney **CHECKED BY:** M. Kwiatkowski

GROUND ELEVATION: ' **GROUNDWATER DEPTH:** Not Encountered **FT. TIME:** **HR.**

ELEVATION (FT)	DEPTH (FT)	SAMPLE TYPE	SAMPLE NO./ CORE RUN NO.	BLOW COUNTS PER 6 IN.	REC (%)	RQD (%)	WC (%)	LL/PL (%)	USCS	qu/Pene (TSF)	STRATA SYMBOL	MATERIAL DESCRIPTION
												Fill - Topsoil
			S-1	3-3-5-9	25							Sandy Gravel (gps), Brn., Dry, Loose
	2											
			S-2	20-20-25-50/3"	50							Weathered Sandstone, Gry., Dense to V. Dense [Auger Refusal @ 6.0']
	4		S-3	50/1"	10							
	6											End of Boring @ 6.0'

SAMPLE LEGEND	GENERAL NOTES
SPT SAMPLE	
SHELBY TUBE	
ROCK CORE	

GAI SELP2 BORING LOGS GPJ 41320

TEST BORING LOG

gai consultants

PROJECT NAME: VA Medical Center - Renovate Basement Clinic Addition	**BORING NO.:** B-3
PROJECT NO.: F121805.00	SHEET 1 OF 1

TOWNSHIP: Wilkes Barre **COUNTY:** Luzerne **STATE:** Pennsylvania

DRILL RIG TYPE: Diedrich D50 Truck **DRILLING METHOD:** HSA **DATE(S) DRILLED:** 02/02/13-02/02/13

DRILLER/COMPANY: T. Turner / Heller Drilling, Inc. **FIELD ENGINEER:** E. Gosney **CHECKED BY:** M. Kwiatkowski

GROUND ELEVATION: ' **GROUNDWATER DEPTH:** Not Encountered **FT. TIME:** **HR.**

ELEVATION (FT)	DEPTH (FT)	SAMPLE TYPE	SAMPLE NO./ CORE RUN NO.	BLOW COUNTS PER 6 IN.	REC (%)	RQD (%)	WC (%)	LL/PL (%)	USCS	qu/Pene (TSF)	STRATA SYMBOL	MATERIAL DESCRIPTION
												Fill - Topsoil
			S-1	2-2-5-7	50							Sandy Gravel (gps), Brn., Dry, Loose
	2		S-2	15-14-16-15	60							Sandy Gravel (gps), Gry., Dry, Dense
	4		S-3	50/4"	20							Weathered Sandstone, Brn., Dry, V. Dense
	6		S-4	50/2"	5							Weathered Sandstone, Lt.-Brn., Dry, V. Dense [Auger Refusal @ 8.0']
	8											End of Boring @ 8.0'

SAMPLE LEGEND	GENERAL NOTES
SPT SAMPLE	
SHELBY TUBE	
ROCK CORE	

GAI SELP2 BORING LOGS GPJ 41320

TEST BORING LOG

gai consultants

PROJECT NAME: VA Medical Center - Renovate Basement Clinic Addition	BORING NO.: B-4
PROJECT NO.: F121805.00	SHEET 1 OF 1

TOWNSHIP: Wilkes Barre **COUNTY:** Luzerne **STATE:** Pennsylvania

DRILL RIG TYPE: Diedrich D50 Truck **DRILLING METHOD:** HSA **DATE(S) DRILLED:** 02/02/13-02/02/13

DRILLER/COMPANY: T. Turner / Heller Drilling, Inc. **FIELD ENGINEER:** E. Gosney **CHECKED BY:** M. Kwiatkowski

GROUND ELEVATION: ' **GROUNDWATER DEPTH:** Not Encountered **FT. TIME:** **HR.**

ELEVATION (FT)	DEPTH (FT)	SAMPLE TYPE	SAMPLE NO./ CORE RUN NO.	BLOW COUNTS PER 6 IN.	REC (%)	RQD (%)	WC (%)	LL/PL (%)	USCS	qu/Pene (TSF)	STRATA SYMBOL	MATERIAL DESCRIPTION
												Fill - Topsoil
		SPT	S-1	3-5-8-50/2"	50							Silty Sand with Gravel (sm), Lt.-Brn., Dry, Med. Dense
	2											Weathered Sandstone, Gry. & Red, Dry, V. Dense [Tr. Iron Staining]
	4	SPT	S-2	50/4"	25							
	6											
	8	SPT	S-3	28-50/12"	25							Weathered Siltstone, Lt.-Brn., Dry, V. Dense [Auger Refusal @ 13.0']
	10		S-4	50/0"	0							
	12											
												End of Boring @ 13.0'

SAMPLE LEGEND	GENERAL NOTES
SPT SAMPLE	
SHELBY TUBE	
ROCK CORE	

GAI SELP2 BORING LOGS.GPJ 41320

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			SYMBOL	TYPICAL NAMES
COARSE-GRAINED SOILS (More than half of material is larger than No. 200 sieve size)	GRAVELS (More than half of coarse fraction is larger than No. 4 sieve size)	CLEAN GRAVELS (Little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines
		GRAVELS WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay
	SANDS (More than half of coarse fraction is larger than No. 4 sieve size)	CLEAN SANDS (little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines
			SP	Poorly graded sands, gravelly sands, little or no fines
		SANDS WITH FINES (Appreciable amount of fines)	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS (More than half of material is smaller than No. 200 sieve size)	SILTS AND CLAYS (Liquid limit less than 50)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
			CL	Inorganic clay of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silty, and organic silty clays of low plasticity
	SILTS AND CLAYS (Liquid limit greater than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity, organic silts
	HIGHLY ORGANIC SOILS		PT	Peat and other highly organic soils

N - The Standard Penetration Test value of the soil, determined in accordance with the methods of ASTM D1586. Reported in blows per ft and normalized to standard drilling equipment and an effective overburden pressure of 2 ksf., the n' value equals the number of hammer blows received by the sampler in advancing over the interval from 6 to 18 in. within a given sampling run.

PARTICLE SIZES

Boulders		->305mm
Cobbles		-76.2mm to 305mm
Gravel-	Coarse	-19.05mm to 76.2mm
	Fine	-4.75mm to 19.05mm
Sand -	Coarse	-2.00mm to 4.75mm
	Medium	-0.425mm to 2.00mm
	Fine	-0.074mm to 0.425mm
Silt		-0.005mm to 0.074mm
Clay		-<0.005mm

COHESIVE SOILS [1.]

Consistency	Unconfined Compressive Strength (psf)	Approximate Range of N
Very Soft	Below 500	0-2
Soft	500-1000	2-4
Medium Stiff	1000-2000	4-8
Stiff	2000-4000	8-15
Very Stiff	4000-8000	15-30
Hard	8000-16000	Over 30

COHESIONLESS SOILS [1.]

Density Classification	Relative Density %	Approximate Range of N
Very loose	0-15	0-4
Loose	16-35	5-10
Medium Dense	36-65	11-30
Dense	66-85	31-50
Very Dense	86-100	Over 50

1. Reference: Soil Mechanics, NA VFAC DM-7.1

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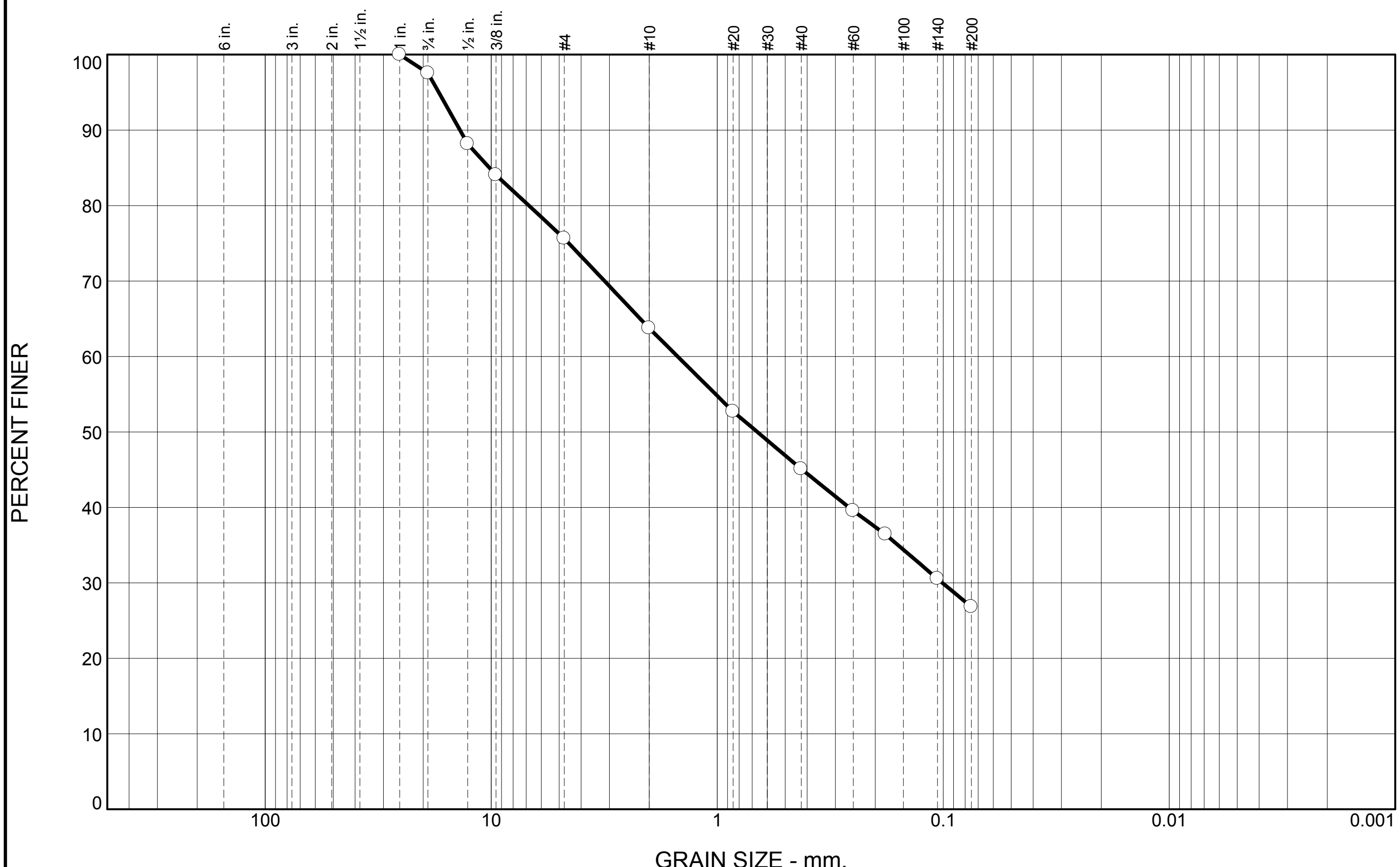
Key to Soil Symbols and Terms

APPENDIX C

Particle Size Distribution Reports (2)

Liquid and Plastic Limits Test Reports
(Natural Water Content Test Results) (2)

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	2.5	21.9	11.9	18.6	18.3	26.8	

	Colloids	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○				10.1804	1.4962	0.6646	0.1008				

Material Description	USCS	AASHTO
○ Brown Silty Sand with Gravel		

Project No. F121805.00 **Client:** VA Medical Center
Project: VA Medical Center - Renovate Basement Clinic Addition
○ **Source of Sample:** B-2 **Depth:** 5.0-5.3 Ft. **Sample Number:** S-3

Date: ○ 2/6/13

Remarks:
○Completed on: 2/8/13

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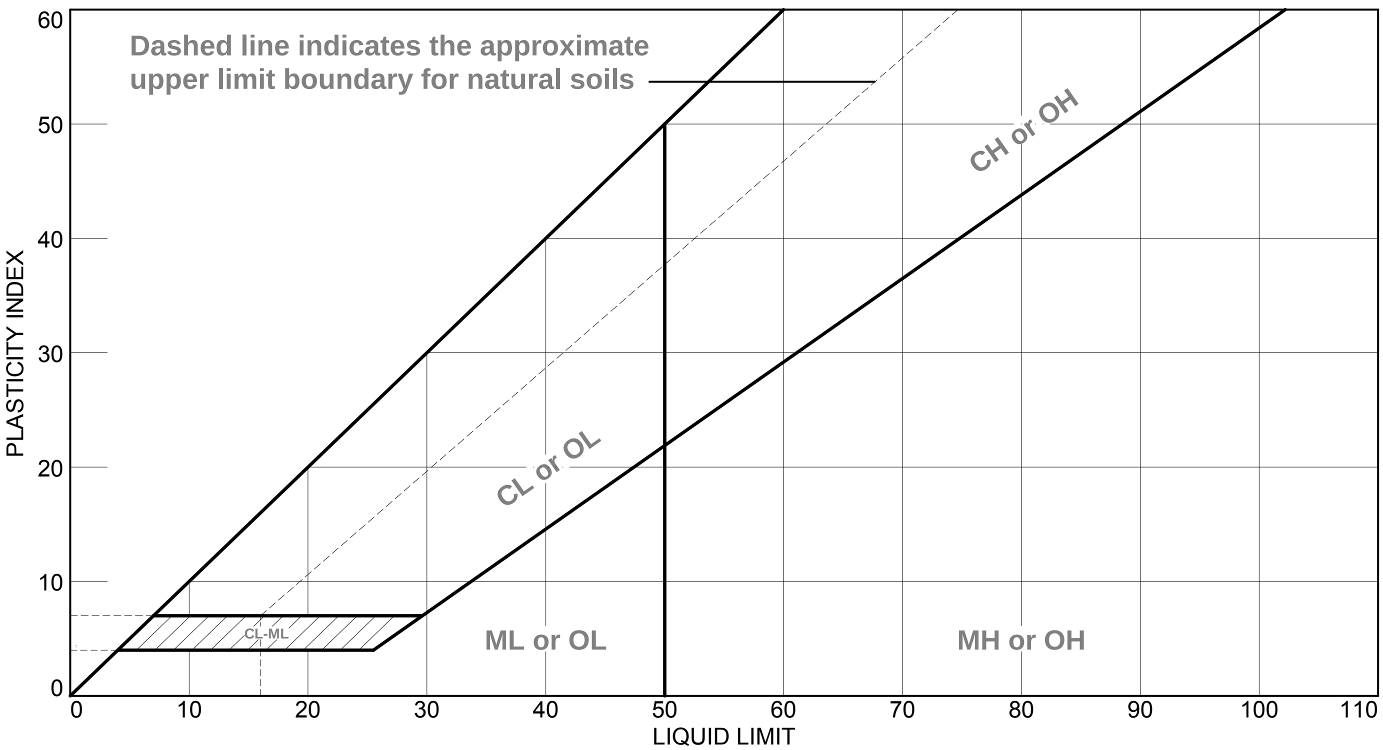
Berwyn, PA

Figure

Tested By: EA **Checked By:** BB

LIQUID AND PLASTIC LIMITS TEST REPORT

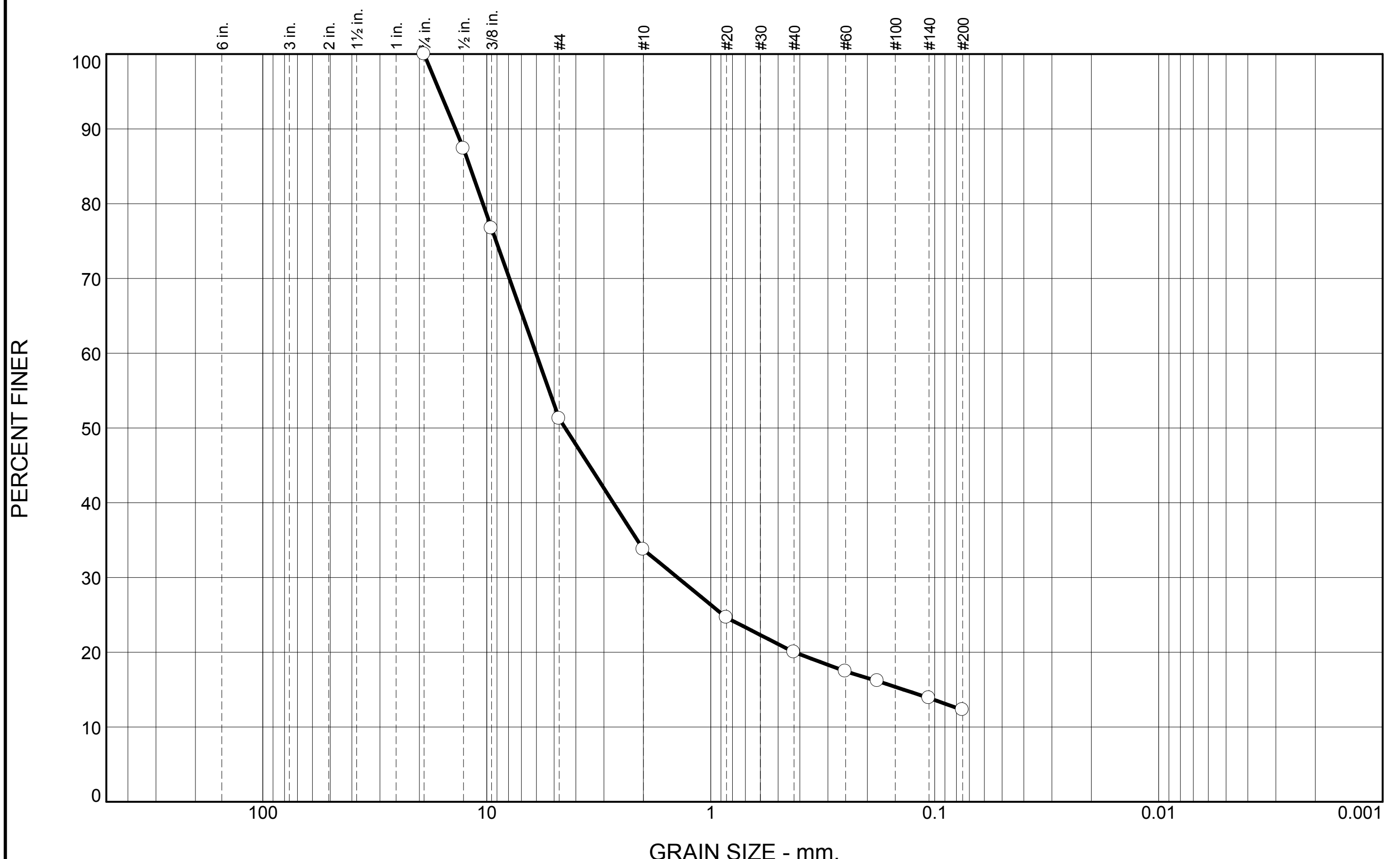


	SOIL DATA							
	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	B-2	S-3	5.0-5.3 Ft.	5.1				

GAI Consultants, Inc. **Berwyn, PA**	**Client:** VA Medical Center **Project:** VA Medical Center - Renovate Basement Clinic Addition **Project No.:** F121805.00 **Figure**

Tested By: EA **Checked By:** BB

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	0.0	48.7	17.6	13.7	7.7	12.3	

	Colloids	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○				11.9128	6.0324	4.4647	1.4078	0.1376			

Material Description	USCS	AASHTO
○ Brown Gray Sandy Gravel with Silt		

Project No. F121805.00 **Client:** VA Medical Center

Project: VA Medical Center - Renovate Basement Clinic Addition

○ **Source of Sample:** B-4 **Depth:** 4.0-4.4 Ft. **Sample Number:** S-2

Date: ○ 2/6/13

GAI Consultants, Inc.

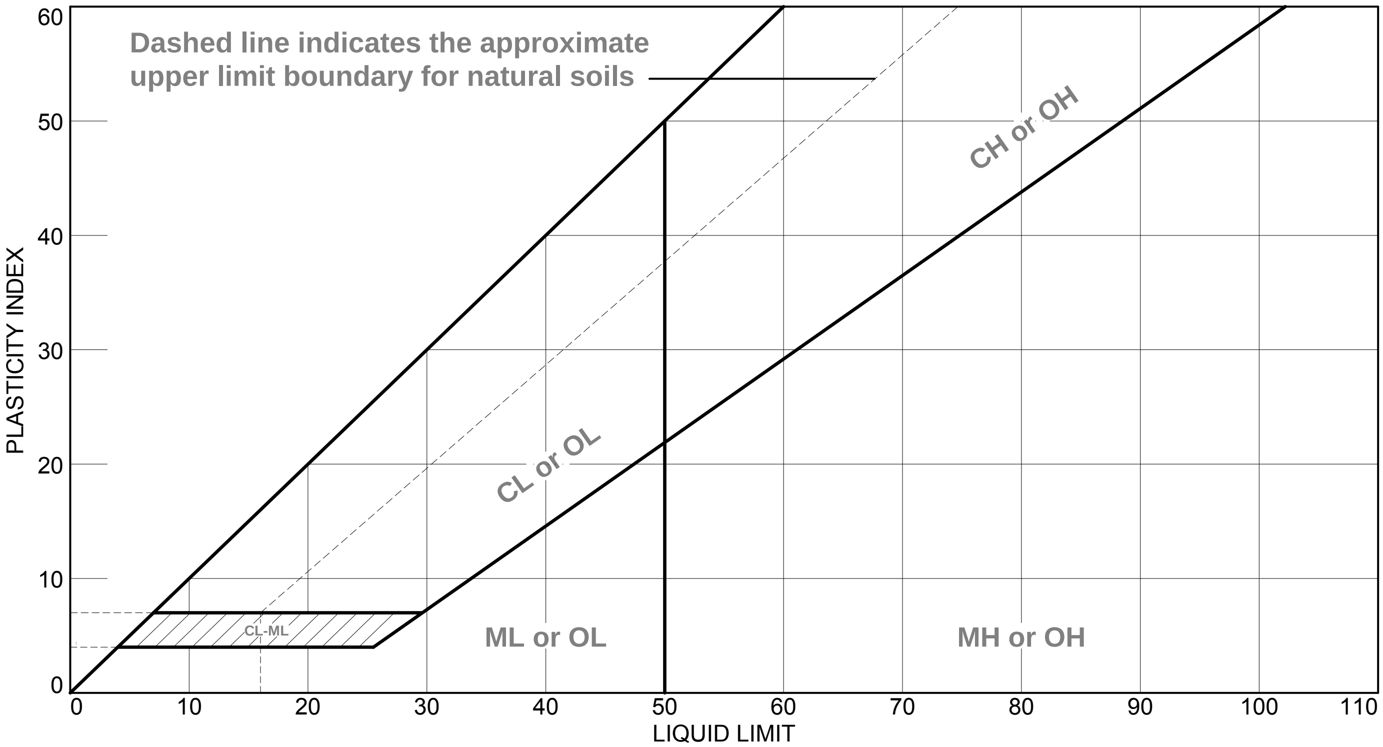
Berwyn, PA

Remarks:
○Completed on: 2/8/13

Figure

Tested By: EA **Checked By:** BB

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA

	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	B-4	S-2	4.0-4.4 Ft.	4.8				

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Client: VA Medical Center
Project: VA Medical Center - Renovate Basement Clinic Addition

Project No.: F121805.00 **Figure**

Tested By: EA **Checked By:** BB