APPENDIX D.4

CONSULTANT REPORTS – GEOTECHNICAL

PRELIMINARY GEOTECHNICAL ENGINEERING REPORT

COPPIN STATE UNIVERSITY PUBLIC SAFETY FACILITY
SITE #1 (N. WARWICK AVE AND PRESBURY STREET)
BALTIMORE, MD
KIM PROJECT NO. G22091

PREPARED FOR
MANNS WOODWARD STUDIOS, INC.
10839 PHILADELPHIA ROAD
WHITE MARSH, MARYLAND 21162

PREPARED BY
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December 12, 2022

Lindsey Kiefer, NCARB, AIA
Project Manager / Architect
Manns Woodward Studios, Inc.
10839 Philadelphia Road
White Marsh, Maryland 21162

Project: **Preliminary Geotechnical Engineering Investigation**
Coppin State University PSF (Site 1)
Baltimore, Maryland
KIM Project No. G21091

Dear Ms. Kiefer,

Kim Engineering Inc. (KIM) is pleased to submit a copy of our report for the above-referenced project. This investigation was conducted in accordance with our agreement dated August 31, 2022.

Services performed include five (5) SPT soil test borings, laboratory testing, and preparation of this preliminary geotechnical investigation report. Our geotechnical services report includes the following:

- An evaluation of the estimated subsurface soil conditions and groundwater conditions at the project site.
- Recommendations for different options of foundations and soil parameters for below-grade walls based on soil test borings and soil laboratory results.
- Seismic site classification information.
- Comments on geotechnical construction aspects that were readily apparent at the time of, in the area of, and to the depth of the investigation.

Services with respect to surveying for line and grade, specific dewatering recommendations, environmental matters, stormwater management recommendations, pavement section design, temporary slopes, seepage analysis, global slope stability analysis, erosion control, cost or quantity estimates, plans, specifications, and construction observation and testing were not included in the scope of services. Soil samples will be held for a period of thirty (30) days after the date of this report and then disposed of, unless an alternate disposition is requested.

We appreciate the opportunity to be of service to you for this project. If you have any questions regarding this project, please do not hesitate to contact either of the undersigned.

Very truly yours,
KIM ENGINEERING, INC.

Kamal Bhusal
Project Manager

Tom Labuda, PE, PG
Principal Engineer

PROFESSIONAL CERTIFICATION: I HEREBY CERTIFY THAT THESE DOCUMENTS WERE PREPARED OR APPROVED BY ME, AND THAT I AM A DULY LICENSED PROFESSIONAL ENGINEER UNDER THE LAWS OF THE STATE OF MARYLAND, LICENSE NO.:PE 42702 EXPIRATION DATE: 10-12-2024.

TABLE OF CONTENTS

1.0 SUMMARY OF CONCLUSIONS AND RECOMMENDATIONS 3
2.0 SITE DESCRIPTION AND PROPOSED CONSTRUCTION 4
3.0 SUBSURFACE EXPLORATION 4
3.1 Test Boring 4
3.2 Rock Core 6
4.0 GEOLOGY 7
5.0 SUBSURFACE CONDITIONS 7
5.1 General Stratification 7
5.2 Groundwater 9
6.0 SOIL GEOTECHNICAL LABORATORY TESTING 10
7.0 PRELIMINARY GEOTECHNICAL ENGINEERING ANALYSIS 11
7.1 Foundation Design Consideration 11
7.1.1 Conventional Spread Footings on Disintegrated Rock or Bedrock 12
7.1.2 Conventional Spread Footings on Impact Rammed Aggregate Piers 12
7.2 Ground Bearing Floor Slab 12
7.3 Seismic Site Coefficient 13
7.4 Below Grade Walls 14
7.5 Permanent Dewatering 15
7.6 Support of Excavation 16
8.0 CONSTRUCTION CONSIDERATIONS 16
8.1 General 16
8.2 Site and Subgrade Preparation 17
8.3 Excavation of Rock 17
8.4 Fill Material and Compaction 18
8.5 Groundwater Control and Site Drainage 18
8.6 Inspection of Subgrades 18
9.0 LIMITATIONS 19

APPENDIX A

Site Location Plan
Boring Location Plan

APPENDIX B

Subsurface Investigation
- Identification of Soil
- Record of Soil/Rock Exploration Logs
- Rock Core photos

APPENDIX C

Geotechnical Laboratory Test Results
- Natural Moisture Contents
- Particle Size Distribution
- Liquid Limit and Plastic Limit Reports

APPENDIX D

Seismic Design Parameters

1.0 SUMMARY OF CONCLUSIONS AND RECOMMENDATIONS

The following is a summary of our conclusions and recommendations:

a. Subsurface conditions in the proposed construction area generally indicate existing fill consisting of Silty Sand, sandy elastic Silt, Gravel with debris in stratum A, underlain by Decomposed Rock and Bedrock in stratum B.

b. Recommended foundation options are presented in section 7.1 of this report. Feasibility of construction will be defined by presence of fill and depth to top of bedrock underlying the site.

c. Compacted fill in structural areas should be classified as silty SAND (SM) or more granular per ASTM D 2487 and compacted to at least 95 percent of maximum dry density per ASTM D 698.

Variations in soil conditions may be encountered during construction. Determination of such variations will permit correlation between the subsurface exploration data of this report and actual conditions encountered during construction and verification of conformance with the plans and specifications. We recommend that Kim Engineering, Inc. be retained to perform professional observations of foundation subgrades.

This report is based on information provided to us on the proposed construction. If the project characteristics are changed from those indicated herein, our recommendations may require modifications. Please advise us of any changes in the proposed construction.

We recommend that the project specifications include the following statement:

> "A preliminary geotechnical report has been prepared for this project by Kim Engineering, Inc. and is available to prospective bidders and/or contractors for informational purposes only. The report has been prepared for design purposes only and may not be sufficient to prepare an accurate bid for construction. Contractors wishing copies of this report may secure them from Kim Engineering Inc. at a nominal charge with the understanding that its scope is limited solely to generalized design considerations."

We have prepared this report in accordance with contemporary geotechnical engineering practices and make no warranties, either expressed or implied, as to the professional services provided under the terms of our agreement and included in this report.

2.0 SITE DESCRIPTION AND PROPOSED CONSTRUCTION

The site is located at Coppin Heights, Baltimore, Maryland and is framed by North Warwick Ave to the East, Baker Street to the South, and Presbury Street to the North. The site consists of open grass covered space. The provided site plan indicates that the site's topography is slightly sloped from a high in the northeast corner to a low in the southwest portion of the site. The surface runoff is in general northeast to southwest direction.

According to the Google Earth historic images the site was occupied by residential rowhouses, paved roadway, and associate facilities before year 2011. The buildings were demolished, and site cleared between 2011 and 2014.

Based on the schematic site plan and information provided to us, the proposed construction will consist of a new 5 story public safety building with 3 below grade parking garage levels, practical Training Village Structure, and associated facilities. The project is in the preliminary stage and detail building plan and structural loads were not provided at the time of writing this report. We understand that the purpose of this subsurface investigation is to determine the feasibility of the site for planned development.

The entire fieldwork was done in readily accessible areas within the proposed construction area as per the boring location provided by the client. The site location plan is appended in Drawing No. 1 in Appendix A.

3.0 SUBSURFACE EXPLORATION

3.1 Test Boring

In order to evaluate the subsurface conditions of the site for the study, a total of five (5) standard penetration tests (SPT) borings (B-1 to B-5) were drilled at the site. The approximate location of the test borings is depicted on the attached Drawing No. 2 in Appendix A (Boring Location Plan).

The standard penetration tests borings were originally planned to extend to 35 feet and 50 feet. All borings were terminated above the planned depths on refusal on rock or fill. Rock cores were obtained in three soil boring locations B-2, B-3, and B-5. The table below summarizes the test boring schedule.

Table 1: Summary of Test Borings

Boring No.	Depth of Boring (ft)	Approximate Existing Ground Elevation (ft) (per site plan)	Depth to Disintegrated Rock (N>60bpf) (ft)	Depth to the Top of Bedrock (ft)
B-1**	21	228	--	--
B-2	37.5*	232	--	27
B-3	56*	228	13.5	41
B-4**	24.2	220	--	--
B-5	41*	228	23.5	31

*Depth with rock Coring
** Auger Refusal in Fill.

The test borings were accomplished using a track mounted drill rig CME-55. The exploration program was performed in the field on November 22nd and 23rd, 2022. Hollow-stem augers were advanced to pre-selected depths and representative soil samples were recovered with a standard split-spoon sampler in general accordance with ASTM D-1586. Disturbed representative soil samples were recovered while performing the Standard Penetration Test (SPT). This test (ASTM D-1586) consists of a 140-pound (lb) hammer falling 30 inches. The number of blows required to drive the standard split spoon sampler (2-inch O.D., 1-3/8-inch I.D.) a distance of 12 inches after an initial set of 6 inches to ensure the sampler is in undisturbed material, is recorded as the Standard Penetration Resistance (N-value) of the soil.

The N-value, for the majority of subsurface situations, provides a generalized indication of in-situ soil conditions when reviewed by individuals with established geotechnical backgrounds. N-values can be used to provide a qualitative indication of the in-place relative density of granular soils. Similarly, N-values provide an indication of consistency for cohesive soils.

Subsurface water level readings were taken in each of the test borings during drilling, at the completion of the drilling process and 24 hours after the drilling process. Upon completion, the boreholes were backfilled with auger cuttings (spoils) and grout on top 10 feet. The backfill material was compacted to the extent feasible; however, some subsidence of the backfill could occur at a future date. As a result, it is recommended that the boreholes be monitored periodically.

Representative portions of the split-spoon soil samples obtained throughout the exploration program were placed in glass jars and transported to our laboratory for further evaluation and visual classification per the visual-manual identification procedure (ASTM D-2488) and the Unified Soil Classification System. The soil descriptions and classifications discussed in this report and shown on the attached boring logs are based on visual observation and as previously noted, should be considered approximate.

Soil samples recovered on this project will be stored at Kim Engineering, Inc. for a period of thirty (30) days from the date of this report. After thirty (30) days, the samples will be discarded unless prior notification for an alternate disposition is provided to us in writing.

3.2 Rock Core

In order to evaluate the bedrock conditions underneath the project site, rock core sampling has been performed in accordance with ASTM D 2113 at the boring locations B-2, B-3, and B-5. Coring of the rock was done in two runs for a total thickness of 10 feet in soil borings B-2 and B-5, and in three runs for a total thickness of 15 feet in B-3 using a diamond bit double core barrel. Color photographs of rock cores are presented in Appendix B.

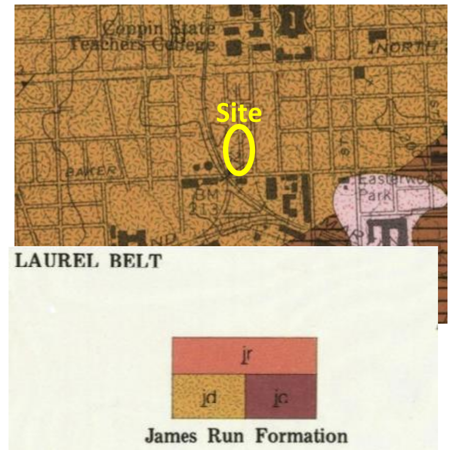
The length of the rock core recovered from a core run has been measured for total core recovery (TCR). Rock quality designation (RQD), which is a modified core recovery percentage in which the lengths of all pieces of sound core over 4-inch long are summed and divided by the length of the core run, has been determined for each core run. The TCR and RQD are summarized in the table below.

Table 2: Summary of Rock Cores

Boring No.	Bedrock Elevation (ft)	1st Run		2nd Run		3rd Run	
		TCR (%)	RQD (%)	TCR (%)	RQD (%)	TCR (%)	RQD (%)
B-2	204.5	60	0	85	0	-	-
B-3	187	60	0	100	0	100	27
B-5	197	100	22	100	42	-	-

4.0 GEOLOGY

According to the "*Geological Map of the Baltimore West Quadrangle, Maryland*" by William P. Crowley and Juergen Reinhardt (1979), the site is underlain mainly by Druid Hill Amphibolite Member (jd) of James Run Formation and described as:



"Fine to medium-grained, generally well foliated amphibolite, locally with irregular anastomosing patches of coarser grained, lighter colored amphibolite. Chlorite fels and actinofels, locally foliated, associated with the amphibolite in places. Includes subordinate quartzo-feldspathic gneiss and granofels to the south which increase northward to nearly half the volume of the unit. Scale of layering ranges from a few tens of centimeters to more than 10 meters. Felsic rocks are generally fine-grained and well foliated, but may also be coarser grained, massive, and intricately jointed."

5.0 SUBSURFACE CONDITIONS

5.1 General Stratification

The subsurface conditions discussed below and those shown on the boring logs represent an estimate of the subsurface conditions based on an interpretation of the boring data using geotechnical engineering judgment. Transitions between different soil strata are usually less distinct than those shown on the boring logs. Although individual test borings are representative of the subsurface conditions at the boring locations on the dates shown, they are not necessarily indicative of subsurface conditions at other locations or at other times.

More comprehensive descriptions of the materials encountered are included in the attached test boring logs. The subsurface investigation indicated that the following generalized strata underlie the site in the areas and to the depths investigated.

Ground Cover:
Borings indicated two (2) to five (5) inches of topsoil beneath the ground surface.

Stratum A (Existing Fill):
Existing Fill was encountered below the ground cover at all the test boring locations. The fill material ranges from the depth of 13.5 feet to 27.5 feet. Auger refusal in fill was encountered in borings B-1 and B-4 at 21 ft and 23.7 ft, respectively. The encountered fill generally consisted of

silty Sand with mica, gravel, asphalt, glass, wood, brick, and concrete fragments. The Standard Penetration Test (SPT) N-values in the existing fill ranged from 5 blows per foot (bpf) to 50 blows per 1 inch. The depth of fill could not be assessed in borings B-1 and B-4. Both borings were terminated in fill due to the auger refusal at the depth listed below. According to the other borings fill can be expected up to the depth of 27.5 feet. The depth for the existing undocumented fill is presented in the following table.

Table 3: Summary of Existing Fill

Boring Identification	Depth of Fill (ft)	Existing Fill Bottom Elevation (ft)
B-1	21*	
B-2	27.5	204.5
B-3	13.5	187
B-4	23.7*	
B-5	23.5	204.5

*Auger Refusal

Stratum B (Disintegrated Rock)

Decomposed rock (Disintegrated Rock), identified as residual material with an N-value greater than 60 bpf, was encountered at various depths in soil borings. This stratum was identified beneath the existing fill in all the boring locations except of B-1 and B-4. The depth to the decomposed/disintegrated rock is provided in table 1.

Disintegrated Rock (also known as decomposed rock) is defined as a residual material with a penetration resistance (N-value) of more than 60 blows per foot and less than refusal (50 blows per 2-inch penetration). It typically retains the remnant rock structure of the parent rock (i.e., is saprolitic) but exhibits the engineering characteristics of a soil when removed. Within a disintegrated rock zone, it is not uncommon to encounter slabs of rock, rock lenses, and/or boulders of intact rock. Also, disintegrated rock levels can vary significantly throughout a particular project site.

It must be stressed that the composition of the disintegrated rock material described on the test boring logs is based on a visual observation of material removed with the auger. In situ materials are very dense rock-like to rock materials. Excavation difficulty as well as specialized excavation techniques should be anticipated in the decomposed rock materials especially in the denser and/or deeper portions of the media.

Auger Refusal on bedrock was recorded in borings B-2, B-3, and B-5 at the depth of 27.5 ft, 41 ft, and 31 ft below the existing ground surface, respectively. Coring of the rock was done in two runs for a total length of 10 feet in soil borings B-2 and B-5, and in three runs for a total length of 15 feet in B-3. The extracted rock was visually examined as gray, streaked and speckled white, light brown, highly fractured, and moderately to highly weathered Gneiss. The general rock quality is poor transitioning to moderate with depth. The description of rock cores is provided in the boring logs appended in Appendix B.

The soil symbols indicated in the stratum descriptions and on the boring logs represent the Unified Soil Classification (ASTM D-2488) group symbols and are based primarily on visual observation of the specimens recovered. Criteria for visual-manual classification of soil samples are given in Appendix B of this report.

5.2 Groundwater

Groundwater observations were performed at the test boring locations. Groundwater was recorded during drilling or at completion of the drilling and 24 hours after the drilling operation. Groundwater was encountered at the depth ranging 10.3 feet to 14.8 feet. The depth for the observed groundwater is presented in the following table.

Table 4: Summary of Groundwater

Boring Identification	Groundwater Readings	
	During Drilling (ft)	24 Hours After Completion of Drilling (ft)
B-1	Dry	11.1
B-2	Dry	13.3
B-3	28.5	10.3
B-4	Dry	14.8
B-5	28.5	12.9

Groundwater level readings are considered to be reliable indication of the water levels at the time indicated. However, fluctuations of groundwater levels as well as perched water may be expected with variations in precipitation, evaporation, surface runoff, and related factors.

6.0 SOIL GEOTECHNICAL LABORATORY TESTING

Geotechnical laboratory testing was performed on jar samples obtained from selected test borings for soil classification, plasticity index, moisture content and standard proctor. Tests were performed in accordance with their associated ASTM Standards. The test results are presented in Appendix C. The associated ASTM methods are presented below:

ASTM Method	Description
D-2216	Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
D-422	Standard Test Method for Particle-Analysis (Grain Size)
D-4318	Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Table 5: Summary of Laboratory Test Results:

Boring Identification	Sample No.	Depths (ft)	Percent Fines (-#200)	Liquid Limit (LL)	Plasticity Index (PI)	Natural Moisture (%)	USCS
B-1	S-5	10.0-11.5	43.1	NV	NP	4.0	SM
B-2	S-4	7.5-9.0	25.3	NV	NP	13.5	SM
B-4	S-7	18.5-20.0	57.8	35	10	11.8	ML
B-5	S-7	18.5-20.0	50.8	52	22	27.4	MH

USCS Soil classification as determined by the Unified Soil Classification System.

LL: Liquid limit: the moisture percentage at which soil behavior transitions from plastic to liquid.

PI: Plastic index: The difference between the plastic and liquid limits (PI = LL – PL), indicates the range of moisture that the soil acts in a plastic manner. The plastic limit is defined as the minimum moisture percentage at which a soil behaves in a plastic manner.

NP Non-Plastic.

NV Non-Viscous

7.0 PRELIMINARY GEOTECHNICAL ENGINEERING ANALYSIS

The following evaluations and recommendations are based on our observations at the site, interpretation of the field data obtained during this exploration, and our experience with similar subsurface conditions and projects. Soil penetration data have been used to estimate an allowable bearing pressure using established correlations. Subsurface conditions in unexplored locations may vary from those encountered.

Determination of an appropriate foundation system for a given structure is dependent on the proposed structural loads, soil conditions, and construction constraints such as proximity to other structures, etc. The subsurface exploration aids the geotechnical engineer in determining the soil stratum appropriate for structural support. This determination includes considerations with regard to both allowable bearing pressure and compressibility of the soil strata. In addition, since the method of construction greatly affects the soils intended for structural support, consideration must be given to the implementation of suitable methods of site preparation, fill compaction and other aspects of construction. The following foundation design criteria are preliminary and provided for planning purposes only. Once the architectural and structural designs are finalized, KIM should review copies of the plans and specifications to revise or expand our recommendations.

7.1 Foundation Design Consideration

Soil profiles encountered across the proposed new construction site were defined by uncontrolled fill up to depths of approximately 27.5 feet consisting of loose to very dense silty Sand, sandy elastic Silt, Gravel with various amounts of deleterious and organic matter. We understand that the proposed structure will be up to 5 stories high and with 3 underground levels. The lowest level floor elevation is planned at approximately (±) 30 feet below the existing ground elevation.

Based on the results of the field subsurface investigation deep fill and depth to bedrock will govern the foundation design. The existing fill is not suitable to support the new building. Conversely, the relatively shallow bedrock will require rock excavation methods for the 3-levels below grade parking garage and basement planned for this project.

Based on or subsurface exploration and our experience with similar subsurface conditions and projects, the following foundation options are proposed for the design.

7.1.1 Conventional Spread Footings on Disintegrated Rock or Bedrock

The existing disintegrated rock or dense natural soil encountered during this exploration are considered suitable for support spread footings. The foundations should be proportioned for a net allowable soil bearing pressure of 6,000 psf when founded on approved natural granular soils of Stratum B or on decomposed rock (N>60 bpf) and net allowable soil bearing pressure of 10,000 psf when founded on competent Bedrock. We do not recommend placing new foundations on the existing fill.

7.1.2 Conventional Spread Footings on Impact Rammed Aggregate Piers

The use of Impact system Rammed Aggregate Piers (RAP), at least 24 inches in diameter are an alternative method to improve the foundation subgrade soils consisting of unsuitable soil and fill. The RAP piers should penetrate through the existing fill and terminate in dense disintegrated rock or on top of the bedrock below.

7.2 Ground Bearing Floor Slab

We do not recommend supporting the concrete slab-on-grade on existing fill. The presence of soft, loose, and organic compressive matter in fill will cause differential settlement and damage to the concrete surface. If the deep fill cannot be safely excavated and replaced with new compacted fill, the structurally supported slab will be required in this location.

For slabs placed on new compacted structural fill we recommend a modulus of subgrade reaction (k) of 125 pounds per cubic inch (pci) for approved subgrades (k value considers a 1-ft by 1-ft square plate). A minimum 6-inch-thick layer of free-draining aggregate is recommended to be placed below the floor slab to serve as a capillary moisture barrier. A polyethylene membrane or similar vapor barrier should be placed over the aggregate to prevent concrete contamination. Proper mix designs, placement methods, and curing methods must be utilized to reduce the potential for concrete shrinkage issues and curling that are sometimes associated with the use of a vapor barrier. Control joints should be provided to control shrinkage cracks of the concrete floor system.

Slab subgrades are often disturbed after final grading due to ongoing construction activities, utility installations, and weather conditions. We recommend that subgrades that become saturated or lose their support capabilities be removed and replaced with new selected compacted engineered fill.

7.3 Seismic Site Coefficient

We are providing a Seismic Site Class Definition per the 2018 International Building Code (IBC) and American Society of Civil Engineers ASCE 7 guidance.

Our scope of services did not include a seismic conditions survey to determine site-specific (accurate) shear wave velocity information. IBC 2018 provides a methodology for interpretation of Standard Penetration Test resistance values (N-values) to determine a Site Class Definition. However, this method requires averaging N- values over the top 100 feet of the subsurface profile, a depth well in excess of the depths of the test borings.

Based on the subsurface data presently obtained and in general accordance with the 2018 IBC, it appears reasonable to assign the site a Classification "D". However, lowering the building foundations to bedrock will allow for higher Classification "C" in design.

The "U.S. Seismic Design Map Web Application" available through the USGS and ASCE websites provides hazard curves, uniform hazard response spectra, and design parameters. These parameters were developed using two percent probability of exceedance (PE) in 50 years. The mapped spectral response acceleration values for the project site are provided in the table below.

Table 6: Mapped Spectral Response Acceleration Values (Class C and D)

Description	Period (Sec)	Sa
Mapped Short Period Spectral Response Acceleration (Ss)	0.2	0.141
Mapped 1-Second Period Spectral Response Acceleration (S_1)	1.0	0.043

For a Site Class C and D, with the above-indicated mapped spectral acceleration values and risk category III, the calculated site coefficient values and the maximum and design spectral response acceleration values are provided in the table below.

Table 7: Site Coefficients, and Design Spectral Response Acceleration (Class C and D)

Soil and Rock Profile	Soil	Dense Soil and Rock
Seismic Site Class	D	C
Site Coefficient (Fa)	1.6	1.3
Site Coefficient (Fv)	2.4	1.5

Short Period, Maximum Spectral Response Acceleration (S_{MS})	0.225	0.183
1.0 Second Period, Maximum Spectral Response Acceleration (S_{M1})	0.104	0.065
Short Period, Design Spectral Response Acceleration (S_{DS})	0.15	0.122
1.0 Second Period, Design Spectral Response Acceleration (S_{D1})	0.069	0.043

Based on our subsurface investigation and engineering judgement, the proposed site is not susceptible to liquefaction under the design earthquake magnitude provided by the code.

7.4 Below Grade Walls

Below-grade walls or basement walls associated with the project should be designed to withstand lateral earth pressures from the backfill and supported soils. Additionally, the walls should be designed to resist the lateral components of surcharge loads occurring within a zone defined by a plane extending up at a 45-degree angle from the base of the wall.

We recommend that the buildup of hydrostatic pressures be precluded by specifying a free-draining fill material immediately adjacent to below-grade walls, with a gravity-driven subdrainage system at the base of the walls.

Earth pressures on walls below grade are influenced by structural design of the walls, conditions of wall restraint, methods of construction and/or compaction and characteristics of the materials being restrained. The most common conditions assumed for earth retaining wall design are the active and at-rest conditions. Active conditions apply to relatively flexible earth retention structures, such as free-standing walls that require rotation and movement to mobilize soil shear strength without affecting their function. Basement walls are rigidly constrained and should be designed utilizing at-rest conditions. A passive condition also exists to represent the maximum possible pressure that may be developed by soils resisting the forces exerted by the active or at-rest conditions. The magnitude of movement required to completely mobilize the passive forces is often beyond aesthetic and/or structural design tolerances in addition to uncertainties during foundation construction, use of passive pressure should be used cautiously, if at all, and be assigned a factor of safety of no less than two (FS>2).

To prevent unforeseen increases in lateral loading, large vehicular and heavy excavation equipment should not operate within a lateral distance equal to the wall height or five (5) feet, whichever is greater. Grading during site development and construction should be maintained to meet the intent of the final design, thus preventing channeled drainage toward partially complete retaining wall structures that could result in delay or damage. This may require diversion dikes, level spreaders, or berms that are not depicted on the erosion and sediment control plan. It is highly recommended that these changes be discussed with the civil design firm to verify that they will not overload storm water management facilities.

The underlying table provides typical parameters for AASHTO #57 crushed stone as well as the encountered on-site soils and import material that might be utilized for the design of retaining structures/walls. The values assigned to the latter are somewhat conservative due to the variable composition of representative samples. Suitable on-site soils would include silty sand (SM) after verification of natural moisture content.

Table 8: Below-Grade Wall Design Parameters

Earth Pressure Condition	AASHTO #57	On-Site Soils [1]
Active (K_A)	0.22	0.33
At-Rest (K_O)	0.36	0.5
Passive (K_P)	4.60	3.0
Moist Unit Weight (γ)	110 pcf	120 pcf
Angle of Internal Friction (ϕ)	40°	30°
Sliding Coefficient (soil-concrete)	0.55	0.35

Note 1: Classified as Silty Sand ("SM"), or better

Use of the parameters assumes that a full-height drainage system has been installed and maintained during construction and throughout the life of the structure. The system should conform to section 1805 of the IBC relating to damp-proofing as groundwater is in excess of five (5) feet below the potential foundation elevation assumed for the project.

7.5 Permanent Dewatering

If the subbase level is not designed for hydrostatic pressure, a permanent dewatering system should be implemented to prevent the groundwater from impacting the structure and to minimize the transmission of moisture through walls. To avoid producing hydrostatic pressures on the sublevel walls, it is recommended that an approved vertical drain be constructed along the entire exterior of the below grade walls. The system would incorporate drain tile in Maryland No. 57 stone enveloped with filter fabric to route the water to sumps and sump pumps.

It is recommended that a subfloor drainage (subdrainage) system be installed below the concrete floor slab of any underground spaces to preclude development of hydrostatic uplift pressure on the lowest level floor slab and to promote a dry space. A subdrainage system consisting of perforated pipe placed in gravel-filled trenches may be installed beneath the slab on grade to control groundwater. Gravel should be wrapped in non-woven drainage filter fabric. The perimeter line may be installed running around the interior perimeters of basement areas with an adequate slope to facilitate efficient water removal and be designed to discharge to sump pit and pump systems. Interior subfloor drainage system and exterior drainage system could be connected with weep holes and or bleeder pipes in order to make flow of water to the sump pit and pump system.

7.6 Support of Excavation

It is anticipated that temporary excavation support will be required during construction. In our opinion, excavation support consisting of soldier piles and timber lagging is considered suitable. Due to the depth of the excavation, soldier piles system with tie backs is appropriate. A soldier pile wall consists of vertical steel beams, typically spaced from 6 to 10 feet apart along the proposed excavation wall, spanned by timber lagging. Prior to the excavation, the steel beams are installed to the designed depth and then backfilled with concrete. Timber lagging is installed between the piles to further stabilize the walls of the excavation. The excavation support should be designed to resist the full earth, water, and surcharge loads acting on it. Surcharge loads from the construction equipment's must be considered. Other additional loads may be required based on the Contractor's planned construction methods.

8.0 CONSTRUCTION CONSIDERATIONS

8.1 General

The principal purpose of this section is to comment in general on the items related to foundation construction, earthwork, and related geotechnical engineering aspects of construction that should be expected for this project. It is recommended that the geotechnical engineer be retained to provide soil engineering services during the actual site preparation and foundation construction phases of the project to perform appropriate evaluations to help ensure that conditions encountered during construction are similar to conditions encountered in the borings. The geotechnical engineer can also assist in interpretation of differing subsurface conditions that may be encountered and recommend remedial work, if needed.

8.2 Site and Subgrade Preparation

Areas proposed for grading or construction should be stripped and grubbed of all existing pavement, topsoil, vegetation, roots, organics, and loose and soft on-site soils before placing structural fill. In addition, existing foundations, abandoned utilities, underground tanks, cisterns, or surface drainage systems such as field tile or perforated pipes possibly encountered in the construction areas should be undercut, removed, or appropriately plugged and backfilled with structural fill in accordance with the recommendations provided in Section 8.4 of this report and at the discretion of a Geotechnical Engineer.

Following preparation of exposed subgrades, accessible portions of the new structure and pavement subgrade should be proof rolled with a loaded 20-ton tandem axle dump truck and witnessed by the Geotechnical Engineer or qualified representative. The purpose of the proof rolling will be to locate any isolated soft, unstable or "pumping" pockets of soil, which should be excavated or otherwise stabilized as directed by the Geotechnical Engineer. Proper site drainage should be maintained at all times to prevent ponding of water at the site during construction. If the soils do become wet, care should be taken to minimize heavy construction equipment from operating on the prone subgrade.

The temporary grades should be sloped at no steeper than 1:5 horizontal to 1 vertical (1.5H:1V). All cleared and grubbed material should be disposed of outside and below the limits of the project area.

8.3 Excavation of Rock

The encountered rock at three borings is characterized by a 0 percent to 42 percent RQD. However, variations in rock quality should be expected across the project site. Heavy duty excavation equipment such as backhoes equipped with rock teeth or bulldozers equipped with ripping attachments can sometimes excavate highly weathered bedrock. However, blasting could be required, if the bedrock is above the basement levels of the proposed building.

For weathered and highly fractured bedrock, there is some potential for localized instability. In such cases, careful inspection during construction and installation of a shoring system is recommended.

8.4 Fill Material and Compaction

The project near-surface soils generally consisted of naturally occurring soils consisting of sandy SILT (ML) and silty SAND (SM). On-site soil i.e., silty SAND (SM) that is free of organic matter or debris, waste materials, frozen materials is considered to be suitable for reuse as compacted engineered fill. Sorting to remove existing fill material and oversized material (larger than 3 inches in diameter) may be required. Proposed fill material that will be subject to third party compaction testing should be subjected to laboratory analysis consisting of, but not necessarily limited to, Proctor moisture/density determination, Atterberg limits, and gradation.

If imported fill is required at the site, we recommend that the material have low expansive characteristics and shall have Unified Soils Classification (ASTM D 2487) of ML or better. Any imported soil fill required to balance the site should adhere to the following parameters unless specifically accepted in writing by the Geotechnical Engineer at time of placement:

Maximum Dry Density (ASTM D698)	$\geq$ 110 pcf
Liquid Limit	$\leq$ 30
Plasticity Index	$\leq$ 15
Expansion Index	$\leq$ 40

We recommend that the fill material be placed in lifts having a maximum loose lift thickness commensurate with the equipment being utilized to perform the compaction. In no case should those lifts exceed eight (8) inches. Each lift should be uniformly compacted to at least 95% of the laboratory maximum dry density as determined by ASTM D 698 based on Baltimore City requirements.

8.5 Groundwater Control and Site Drainage

Based upon the borings, groundwater will be encountered during construction. Installation of a perimeter construction dewatering system may be required for deep excavation. The system selection, design, and testing should be provided by a specialty dewatering contractor with local practice of at least 5 years.

8.6 Inspection of Subgrades

We recommend that all subgrades be inspected by a Geotechnical Engineer or an experienced engineering technician. Subgrades should be tested to check whether any unstable areas exist. Any unstable zones that are identified that cannot be re-compacted should be undercut to a depth, within the area marked by the inspecting engineer. The depths and extent of undercuts should be determined by the inspecting Geotechnical Engineer. Deeper undercuts should be avoided, and it is requested that KIM be extended an opportunity to review the conditions warranting any deeper undercuts before undercutting commences. Undercut volume should be backfilled to grade with compacted fill in accordance with the requirements in this report.

9.0 LIMITATIONS

This report has been prepared for the exclusive use by our client for specific application to the proposed construction as presented herein. Our services were performed in accordance with contemporary soil and foundation engineering practices. No warranty, either expressed or implied, is made. Our conclusions and recommendations are based on the preliminary design information furnished to us, the data obtained from the subsurface exploration program, and/or current geotechnical engineering practices. The findings and recommendations do not reflect variations in subsurface conditions that could exist between the boring locations or in unexplored areas of the site. Should such variations become apparent during construction, it will be necessary to re-evaluate our conclusions and recommendations based upon on-site observations of the conditions.

Regardless of thoroughness of a subsurface exploration, there is the possibility that conditions in other areas will differ from those at the boring locations and the conditions may not be as anticipated by the designers. Additionally, the construction process may alter the soil conditions. Therefore, experienced geotechnical engineers should evaluate earthwork and foundation construction to verify that the conditions anticipated in design actually exist in the field at the time of construction. Otherwise, we assume no responsibility for construction compliance with the design concepts, specifications, or recommendations.

In the event that changes are made in the design or location of the proposed facilities, the recommendations presented in the report shall not be considered valid unless the changes are reviewed by our firm and conclusions of this report modified and/or verified in writing.

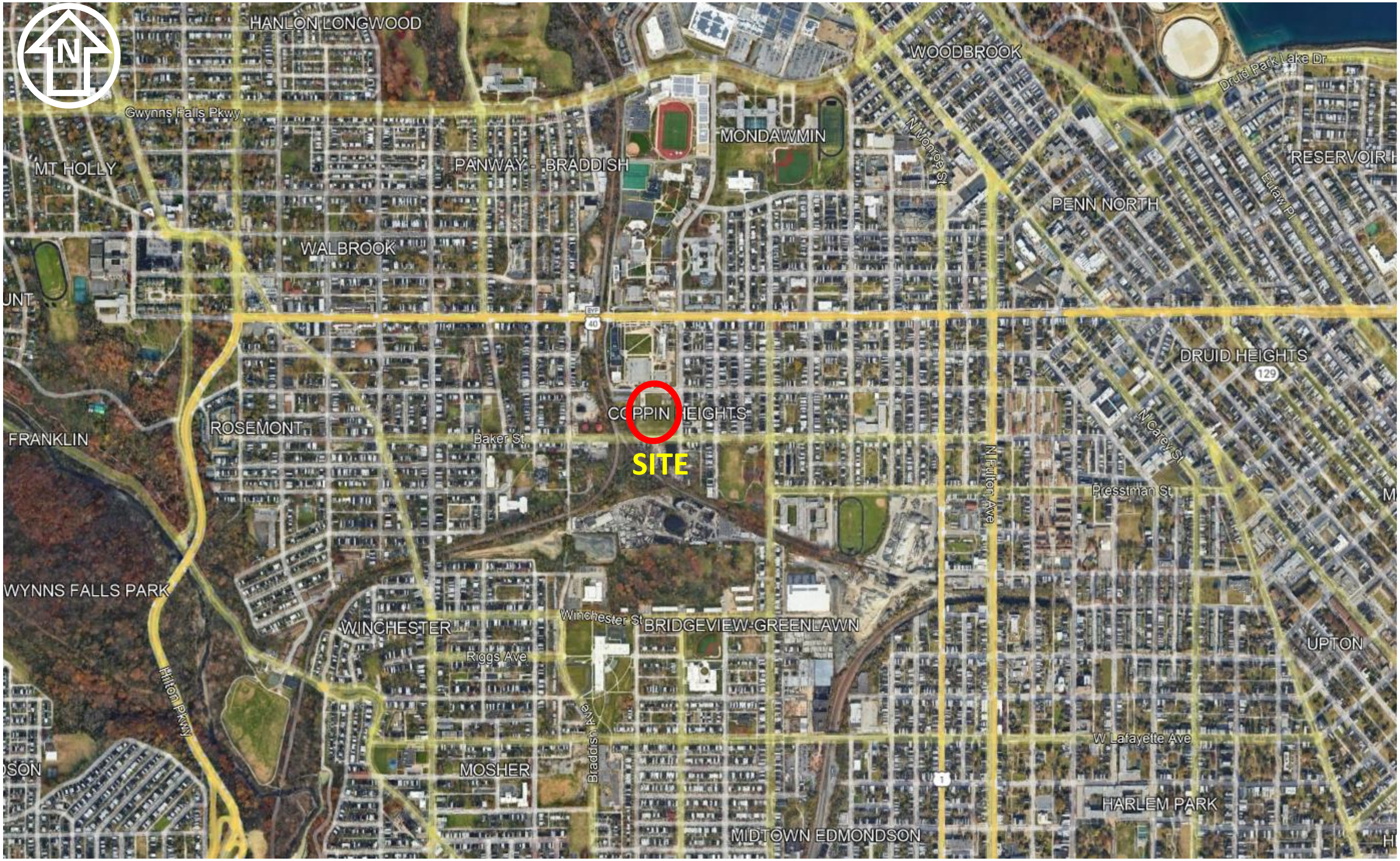
If this report is copied or transmitted to a third party, it must be copied or transmitted in its entirety, including text, attachments, and enclosures. Interpretations based on only a part of this report may not be valid.

It is important to note that our study was done in an effort to assist planning and design personnel in the preparation of generalized drawings and specifications for the project. As a result of this, potential contractors should be encouraged to conduct their own individually tailored studies to assess soils conditions, rock levels, excavation slope gradients, temporary excavation support methods, and groundwater/perched water levels and conditions. Specifically, our report has been prepared for generalized purposes of planning and design and may not be sufficiently comprehensive for bid preparation purposes.

APPENDIX A

Site Location Plan

Boring Location Plan



Sourced by Google Map

KIM ENGINEERING

3916 VERO ROAD, SUITE K BALTIMORE, MD 21227

SITE LOCATION PLAN

COPPIN STATE UNIVERSITY PUBLIC SAFETY FACILITY

SITE # 1 SITE #1 (N. WARWICK AVE AND PRESBURY STREET)

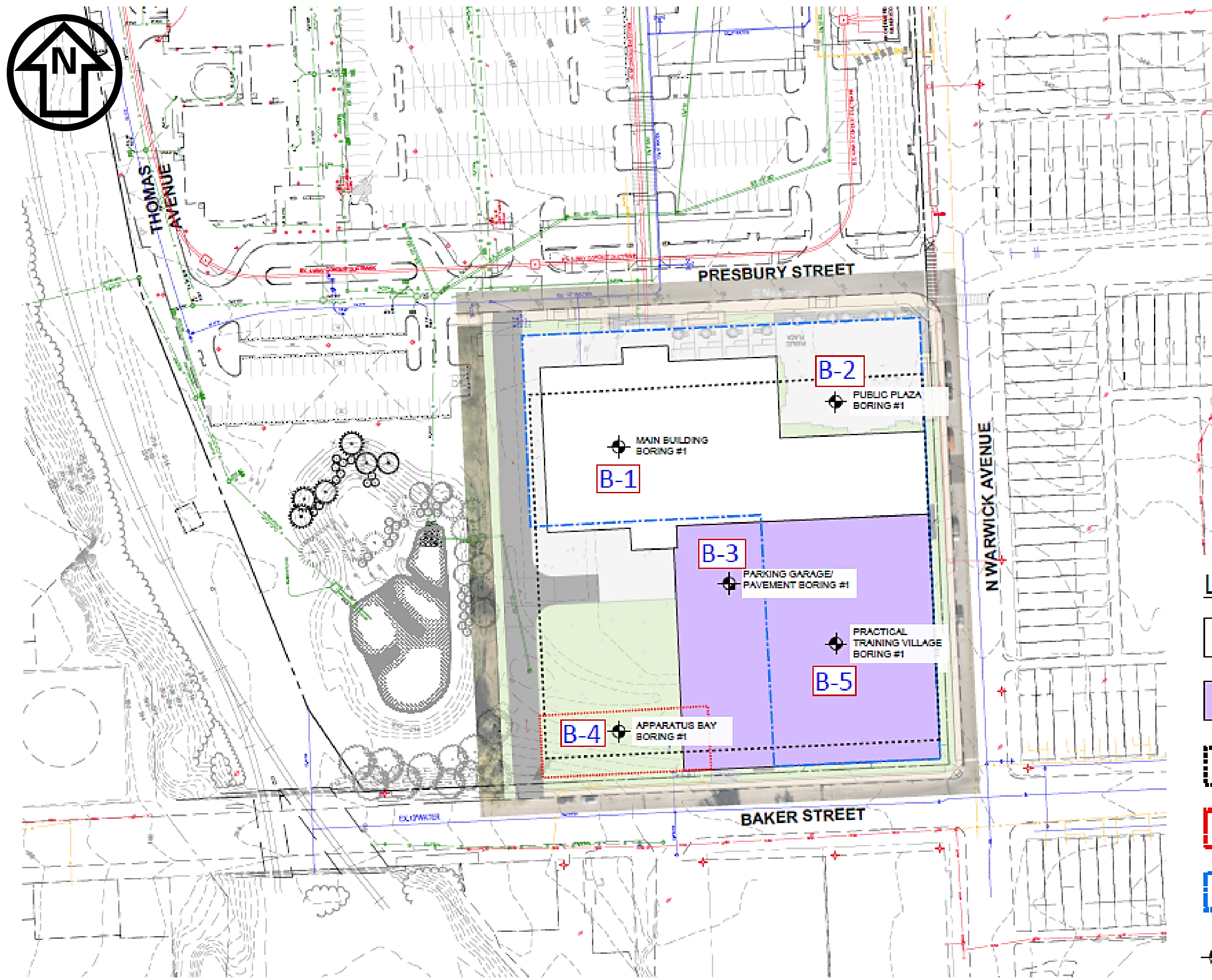
BALTIMORE, MD

PROJECT NO.: G22091

SCALE: NTS

DATE: DECEMBER 8, 2022

DRAWING NO. 1



EX. ZONING: EC-2
EDUCATIONAL CAMPUS ZONING. A CAMPUS DISTRICT FOR COLLEGES AND UNIVERSITIES THAT ALLOWS FOR CERTAIN NON-EDUCATIONAL USES AND DORMITORIES FOR STUDENTS.

REQUIRED SETBACKS
NONE IF BUILDING DOES NOT ABUT ANOTHER ZONING DISTRICT OTHERWISE:
FRONT - 20'
INTERIOR SIDE - 10'
CORNER SIDE - 20'
REAR - 15'

REQUIRED PARKING FOR POST SECONDARY EDUCATIONAL FACILITY:
1 PER 4 EMPLOYEES ON PEAK SHIFT +
1 PER 25 STUDENTS BASED ON THE MAXIMUM STUDENT CAPACITY

LEGEND

- MAIN BUILDING STRUCTURE
  5 ABOVE GROUND STORIES
- PRACTICAL TRAINING VILLAGE STRUCTURE
  4 ABOVE GROUND STORIES
- PARKING GARAGE STRUCTURE
  3 BELOW GROUND STORIES
- APPARATUS BAY STRUCTURE
  1 ABOVE GROUND STORY
- FIRING RANGE STRUCTURE
  1 BELOW GROUND STORY
- BORING LOCATIONS

KIM ENGINEERING
3916 VERO ROAD, SUITE K BALTIMORE, MD 21227

BORING LOCATION PLAN
COPPIN STATE UNIVERSITY PUBLIC SAFETY FACILITY
SITE # 1 SITE #1 (N. WARWICK AVE AND PRESBURY STREET)
BALTIMORE, MD

PROJECT NO.: G22091
SCALE: NTS
DATE: DECEMBER 8, 2022
DRAWING NO. 2

APPENDIX B

SUBSURFACE INVESTIGATION

Identification of Soil

Record of Soil/Rock Exploration Logs

Rock Core Photos

IDENTIFICATION OF SOIL

Soil Classification - ASTM D-2487

Coarse Grained Soils, More than 50% is retained on the No. 200 sieve	**Gravels -** More than 50% of the course fraction is retained on the No. 4 sieve. Coarse = 1" - 3" Medium = 1/2" - 1 " Fine = 1/4" to 1/2"	Clean Gravels <5% Passing No. 200 sieve	**GW**	Well Graded Gravel
			GP	Poorly Graded Gravel
		Gravels with fines >12% passing No. 200 sieve	**GM**	Silty Gravel
			GC	Clayey Gravel
	Sands - More than 50% of the coarse fraction passes the No.4 sieve Coarse = No. 10 to No. 4 Medium = No. 10 to No. 40 Fine = No. 40 to No. 200	Clean Sands <5% passing No. 200 sieve	**SW**	Well Graded Sand
			SP	Poorly Graded Sand
		Sands with fines >12% passing No. 200 sieve	**SM**	Silty Sand
			SC	Clayey Sand
Fine Grained Soils, More than 50% passes the No. 200 sieve	**Silts and Clays** Liquid Limit of 50 or less Low to medium plasticity	Inorganic	**ML**	Silt
			CL	Lean Clay
		Organic	**OL**	Organic silt
				Organin clay
	Silts and Clays **L**iquid limit of 50 or greater Medium to high plasticity	Inorganic	**MH**	Elastic silt
			CH	Fat clay
		Organic	**OH**	Organic silt
				Organic clay
Highly Organic	Primarily Organic matter, dark color, organic odor		**PT**	Peat

Terminology and Definitions

Portions of Soil Components		
Component Form	Description	Label
Noun	Gravel, Sand, Silt, Clay	50% or more
Adjective	Sandy, Silty, Clayey	35% to 49%
Some	some Sand, some Silt	12% to 34%
Trace	trace Sand, trace Clay	1% to 11%
With	with Sand, with Silt	presence only

Particle Size Identification	
Particle Size	Particle Dimension
Boulder	12" diamter or more
Cobble	3" to 12" diamter
Gravel	1/4" to 3" diamter
Sand	0.005" to 1/4" diamter
Silt/ Clay (fines)	Cannot See Particle

Cohesive Soils		
Field Description	N- Value	Consistency
Easily Molded in Hands	0-2	Very Soft
Easily Penetrated Several inches by thumb	3-4	Soft
Penetrated by thumb with Moderate Effort	5-8	Medium Stiff
Penetrated by Thumb with Great Effort	9-15	Stiff
Indented by Thumb with only Great Effort	16-30	very Stiff
Difficult to indent by thumbnail	> 30	Hard

Granular Soils	
N- Values	Relative Density
0-4	Very Loose
5-10	Loose
11-30	Medium Dense
31-50	Dense
Greater than 50	Very Dense

Fill: Man made deposit of soils, rock and waste material.
Probable Fill: Soils which contain no visually detected foreign matter but which may be man made deposit.
Rock Fragments: Angular Pieces of rock, distinguished from transported gravel, which have seperated from orginal wein or strata and are present in soil matrix.
Disintregrated Rock: Residual rock material with SPT of more than 60 blows per ft. and less than refusal.
Karst: Descriptive term which denotes the potential for solutioning of limestone rock and the development of sink holes.
Alluvium: Recently depositied soils placed by water action, typically stream or river flood plain soils.
Ironite: Iron oxide deposited within a soil layer forming cemented deposits.
Quarts: A hard silica mineral often found in residual soils.
Mica: A soft plate of silica mineral found in many rocks. And in residual or transported soil derived there from.
Layers: 1/2 to 12 inch seam of minor soil component.
Lenses: 0 to 1/2 inch seam of minor soil component.
Pocket: Discontinuous body of minor soil component.

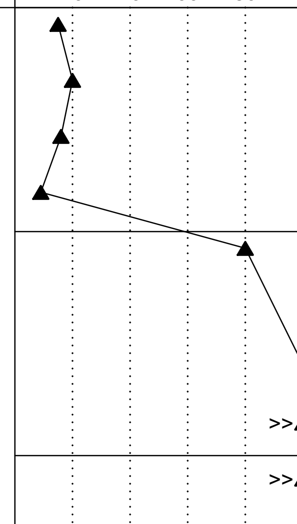
Record of Soil/Rock Exploration Logs

BORING NUMBER B-1 (Site A)

KIM ENGINEERING, INC.
Consulting Geotechinical Engineers
Baltimore, Maryland

CLIENT Manns Woodward Studios, Inc.
PROJECT NAME Coppin State University PSF
PROJECT NUMBER G22091
PROJECT LOCATION Baltimore, Maryland
DATE STARTED 11/22/22 **COMPLETED** 11/22/22
GROUND ELEVATION 228 ft **HOLE SIZE** 6"
DRILLING CONTRACTOR Kim Engineering Inc.
GROUND WATER LEVELS:
DRILLING METHOD H.S.A.
AT TIME OF DRILLING Dry
LOGGED BY SE **CHECKED BY** TL
AT END OF DRILLING Dry
NOTES
24hrs AFTER DRILLING 11.1 ft / Elev 216.9 ft

DEPTH (ft)	GRAPHIC LOG	ELEVATION	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ PL MC LL ☐ FINES CONTENT (%) ☐ 20 40 60 80
		227.58	5-inches of Topsoil.	SS 1	33	4-8-7 (15)			
			Brown, grayish brown, red, gray, greenish gray, light brown, tan, moist silty Sand with concrete, brick and decomposed rock fragments. (FILL)	SS 2	67	5-7-13 (20)			
				SS 3	72	4-7-9 (16)			
				SS 4	56	4-4-5 (9)			
10				SS 5	78	14-41-39 (80)			
				SS 6	83	14-50/6"			
20				SS 7	100	50/1"			>>
		207.00	Rock Fragments/ Gravel.	SS 8	100	50/1"			>>
			Bottom of hole at 21.0 feet.						

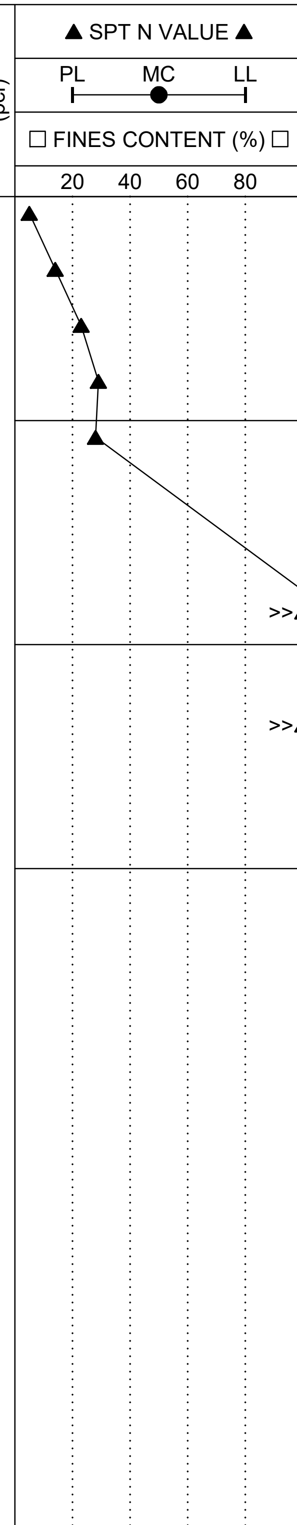


KIM ENGINEERING, INC.
Consulting Geotechinical Engineers
Baltimore, Maryland

BORING NUMBER B-2 (Site A)

CLIENT Manns Woodward Studios, Inc.
PROJECT NAME Coppin State University PSF
PROJECT NUMBER G22091
PROJECT LOCATION Baltimore, Maryland
DATE STARTED 11/23/22 **COMPLETED** 11/23/22
GROUND ELEVATION 232 ft **HOLE SIZE** 6"
DRILLING CONTRACTOR Kim Engineering Inc.
GROUND WATER LEVELS:
DRILLING METHOD H.S.A.
AT TIME OF DRILLING Dry
LOGGED BY SE **CHECKED BY** TL
AT END OF DRILLING ---
NOTES
24hrs AFTER DRILLING 13.3 ft / Elev 218.8 ft

DEPTH (ft)	GRAPHIC LOG	ELEVATION	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)
		231.84	2-inches of Topsoil.					
			Dark brown, greenish gray, brown, dark gray, moist, silty Sand with concrete and brick fragments. (FILL)	SS 1	33	3-2-3 (5)		
				SS 2	67	7-6-8 (14)		
				SS 3	78	7-11-12 (23)		
10				SS 4	78	6-11-18 (29)		
				SS 5	56	8-11-17 (28)		
				SS 6	67	19-50/6"		
20			Gravel and concrete fragments.	SS 7	100	50/1"		
				SS 8	100	50/2"		
30		204.50	Brown, gray, speakled and streaked white, fine to medium grained, highly fractured, highly weathered Gneiss.	RC 1	60 (0)			
		199.50	Brown, gray, speakled and streaked white, fine to medium grained, highly fractured, highly weathered Gneiss.	RC 2	85 (0)			
		194.50	Bottom of hole at 37.5 feet.					

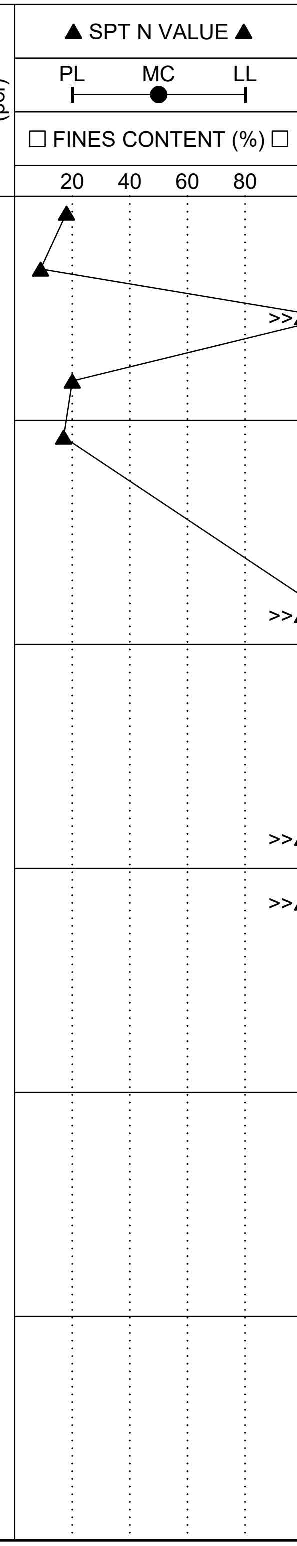


KIM — KIM ENGINEERING, INC.
Consulting Geotechinical Engineers
Baltimore, Maryland

BORING NUMBER B-3 (Site A)

CLIENT Manns Woodward Studios, Inc.
PROJECT NAME Coppin State University PSF
PROJECT NUMBER G22091
PROJECT LOCATION Baltimore, Maryland
DATE STARTED 11/22/22 **COMPLETED** 11/22/22
GROUND ELEVATION 228 ft **HOLE SIZE** 6"
DRILLING CONTRACTOR Kim Engineering Inc.
GROUND WATER LEVELS:
DRILLING METHOD H.S.A.
AT TIME OF DRILLING 28.5 ft / Elev 199.5 ft
LOGGED BY SE **CHECKED BY** TL
AT END OF DRILLING 27.5 ft / Elev 200.5 ft
NOTES
24hrs AFTER DRILLING 10.3 ft / Elev 217.8 ft

DEPTH (ft)	ELEVATION	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)
0	227.67	4-inches of Topsoil.	SS 1	89	4-8-10 (18)		
		Dark brown, dark gray, light brown, gray, black, moist, silty Sand with concrete and brick fragments. (FILL)	SS 2	44	2-4-5 (9)		
			SS 3	100	12-50/4"		
			SS 4	100	10-9-11 (20)		
10			SS 5	50	12-9-8 (17)		
	214.50	DECOMPOSED ROCK classified as brown, dark brown, orange, light brown, moist to wet, very dense, silty SAND (SM) with decomposed rock fragments.	SS 6	92	29-50/6"		
20			SS 7	100	50/5"		
			SS 8	100	38-50/6"		
30			SS 9	100	50/4"		
			SS 10	100	50/1"		
40	187.00	Brown, white, gray, medium to fine grained, highly weathered, highly fractured Gneiss.	RC 1	46 (0)			
	182.00	Gray, speakled and streaked white, fine grained, slightly weathered, highly fractured Gneiss.	RC 2	85 (0)			
50	177.00	Gray, speakled and streaked white, fine grained, slightly weathered, moderately to highly fractured Gneiss.	RC 3	100 (25)			
	172.00	Bottom of hole at 56.0 feet.					



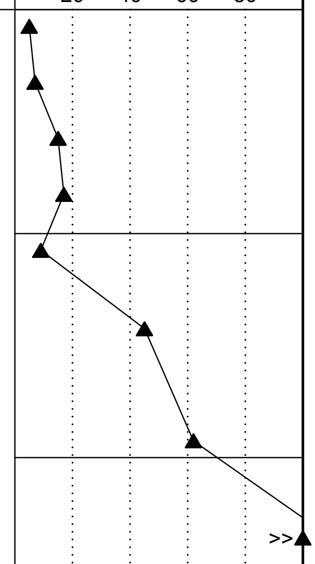
KIM ENGINEERING, INC.
Consulting Geotechinical Engineers
Baltimore, Maryland

BORING NUMBER B-4 (Site A)

CLIENT Manns Woodward Studios, Inc.
PROJECT NAME Coppin State University PSF
PROJECT NUMBER G22091
PROJECT LOCATION Baltimore, Maryland
DATE STARTED 11/22/22 **COMPLETED** 11/22/22
GROUND ELEVATION 220 ft **HOLE SIZE** 6"
DRILLING CONTRACTOR Kim Engineering Inc.
GROUND WATER LEVELS:
DRILLING METHOD H.S.A.
AT TIME OF DRILLING Dry
LOGGED BY SE **CHECKED BY** TL
AT END OF DRILLING Dry
NOTES
24hrs AFTER DRILLING 14.8 ft / Elev 205.3 ft

DEPTH (ft)	GRAPHIC LOG	ELEVATION	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)
		219.67	4-inches of Topsoil.	SS 1	33	3-2-3 (5)		
			Brown, gray, red, moist, silty Sand with gravel, brick and concrete fragments. (FILL)	SS 2	39	3-3-4 (7)		
				SS 3	67	9-9-6 (15)		
10				SS 4	33	6-6-11 (17)		
				SS 5	44	4-5-4 (9)		
			clayey Sand.	SS 6	89	16-20-25 (45)		
20			sandy Silt.	SS 7	89	22-38-24 (62)		
		196.30	Gravel and Concrete fragments. Bottom of hole at 23.7 feet.	SS 8	50	50/2"		

▲ SPT N VALUE ▲
PL MC LL
☐ FINES CONTENT (%) ☐
20 40 60 80

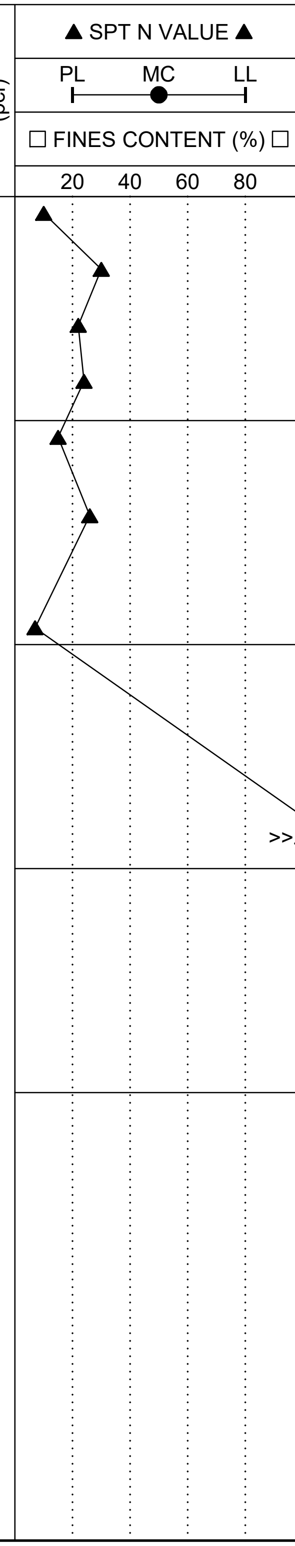


BORING NUMBER B-5 (Site A)

KIM ENGINEERING, INC.
Consulting Geotechinical Engineers
Baltimore, Maryland

CLIENT Manns Woodward Studios, Inc.
PROJECT NAME Coppin State University PSF
PROJECT NUMBER G22091
PROJECT LOCATION Baltimore, Maryland
DATE STARTED 11/23/22 **COMPLETED** 11/23/22
GROUND ELEVATION 228 ft **HOLE SIZE** 6"
DRILLING CONTRACTOR Kim Engineering Inc.
GROUND WATER LEVELS:
DRILLING METHOD H.S.A.
AT TIME OF DRILLING 28.5 ft / Elev 199.5 ft
LOGGED BY SE **CHECKED BY** TL
AT END OF DRILLING Dry
NOTES
24hrs AFTER DRILLING 12.9 ft / Elev 215.1 ft

DEPTH (ft)	GRAPHIC LOG	ELEVATION	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)
0		227.75	3-inches of Topsoil.					
			Dark brown, dark gray, red, black, moist, silty Sand with wood and brick fragments. (FILL)	SS 1	67	3-5-5 (10)		
				SS 2	56	7-16-14 (30)		
				SS 3	67	6-9-13 (22)		
				SS 4	78	10-11-13 (24)		
10				SS 5	78	8-8-7 (15)		
				SS 6	67	8-12-14 (26)		
20			Brown, gray, dark gray, sandy elastic Silt with wood fragments.	SS 7	78	3-3-4 (7)		
		204.50	DECOMPOSED ROCK classified as brown, dark gray, gray, moist to wet, silty SAND (SM) with decomposed rock fragments.	SS 8	300	50/6"		
30				SS 9	100	50/2"		
		197.00	Brown, gray, speakled and streaked white, fine to medium grained, highly fractured, moderately to highly weathered Gneiss.	RC 1	100 (22)			
40		192.00	Brown, gray, speakled and streaked white, fine to medium grained, highly fractured, moderately to highly weathered Gneiss.	RC 2	100 (42)			
		187.00	Bottom of hole at 41.0 feet.					



Rock Core Photos

➢ Photo of Core Box

B-2	1st Run (Top Row) (27.5’ to 32.5’)
	2nd Run (Second Row) (32.5’ to 37.5’)

B-3	1st Run (Top Row) (41’ to 46’)
	2nd Run (Second Row) (46’ to 51’)
	3rd Run (Third Row) (51’ to 56’)

B-5	1st Run (Top Row) (31’ to 36’)
	2nd Run (Second Row) (36’ to 41’)

APPENDIX C

GEOTECHNICAL LABORATORY TESTS

Natural Moisture Contents

Particle Size Distribution Report

Liquid Limit and Plastic Limit Report

Natural Moisture Contents

KIM

KIM ENGINEERING, INC.

Moisture Content Determination

Project Name:	Coppin State University PSF (Site 1)	**Tested By:**	KS
Project No.:	G19064	**Tested Date:**	12/8/2022

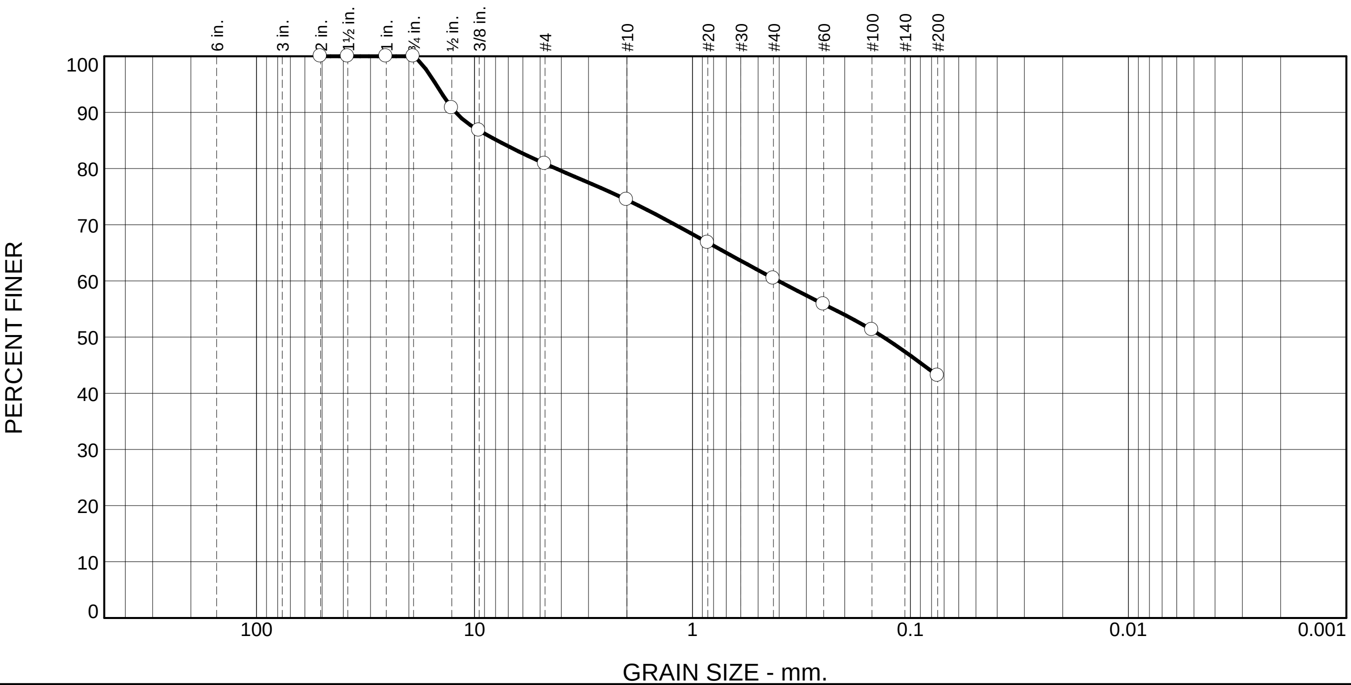
Boring No.	**B-1**				**B-2**		
Depth	**10.0-11.5**	**13.5-15.0**	**18.5-20.0**	**23.5-25.0**	**10.0-11.5**	**13.5-15.0**	**18.5-20.0**
Sample No.	**S-5**	**S-6**	**S-7**	**S-8**	**S-5**	**S-6**	**S-7**
Wt. (wet+tare)	110.68	88.10	390.48	91.86	110.63	454.58	454.58
Wt. (dry+tare)	98.58	78.72	365.50	80.75	104.00	422.38	422.38
Wt. (tare)	21.49	15.79	116.44	18.97	15.68	113.42	113.42
Wt. (water)	12.10	9.38	24.98	11.11	6.63	32.20	32.20
Wt. (dry)	77.09	62.93	249.06	61.78	88.32	308.96	308.96
Moisture Content	**15.70**	**14.91**	**10.03**	**17.98**	**7.51**	**10.42**	**10.42**

Boring No.	**B-3**				**B-4**			
Depth	**10.0-11.5**	**13.5-15.0**	**18.5-20.0**	**23.5-25.0**	**10.0-11.5**	**13.5-15.0**	**18.5-20.0**	**23.5-25.0**
Sample No.	**S-5**	**S-6**	**S-7**	**S-8**	**S-5**	**S-6**	**S-7**	**S-8**
Wt. (wet+tare)	110.68	88.10	390.48	91.86	110.63	454.58	454.58	454.58
Wt. (dry+tare)	98.58	78.72	365.50	80.75	104.00	422.38	422.38	422.38
Wt. (tare)	21.49	15.79	116.44	18.97	15.68	113.42	113.42	113.42
Wt. (water)	12.10	9.38	24.98	11.11	6.63	32.20	32.20	32.20
Wt. (dry)	77.09	62.93	249.06	61.78	88.32	308.96	308.96	308.96
Moisture Content	**15.70**	**14.91**	**10.03**	**17.98**	**7.51**	**10.42**	**10.42**	**10.42**

Boring No.	**B-5**			
Depth	**10.0-11.5**	**13.5-15.0**	**18.5-20.0**	**23.5-25.0**
Sample No.	**S-5**	**S-6**	**S-7**	**S-8**
Wt. (wet+tare)	110.68	88.10	390.48	91.86
Wt. (dry+tare)	98.58	78.72	365.50	80.75
Wt. (tare)	21.49	15.79	116.44	18.97
Wt. (water)	12.10	9.38	24.98	11.11
Wt. (dry)	77.09	62.93	249.06	61.78
Moisture Content	**15.70**	**14.91**	**10.03**	**17.98**

Particle Size Distribution Report

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	19.1	6.5	13.9	17.4	43.1	

TEST RESULTS

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
2"	100.0		
1 1/2"	100.0		
1"	100.0		
3/4"	100.0		
1/2"	90.8		
3/8"	86.8		
#4	80.9		
#10	74.4		
#20	66.8		
#40	60.5		
#60	55.9		
#100	51.3		
#200	43.1		

* (no specification provided)

Material Description

silty sand with gravel (SM)

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= SM **AASHTO (M 145)=** A-4(0)

Coefficients

D_{90}= 12.2022 D_{85}= 7.8695 D_{60}= 0.4032
D_{50}= 0.1329 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 11/22/2022 **Date Tested:** 12/08/2022

Tested By: KS

Checked By: TL

Title: Principal Engineering

Source of Sample: B-1 **Depth:** 10'00"-11'6" **Date Sampled:** 11/22/2022
Sample Number: S-5

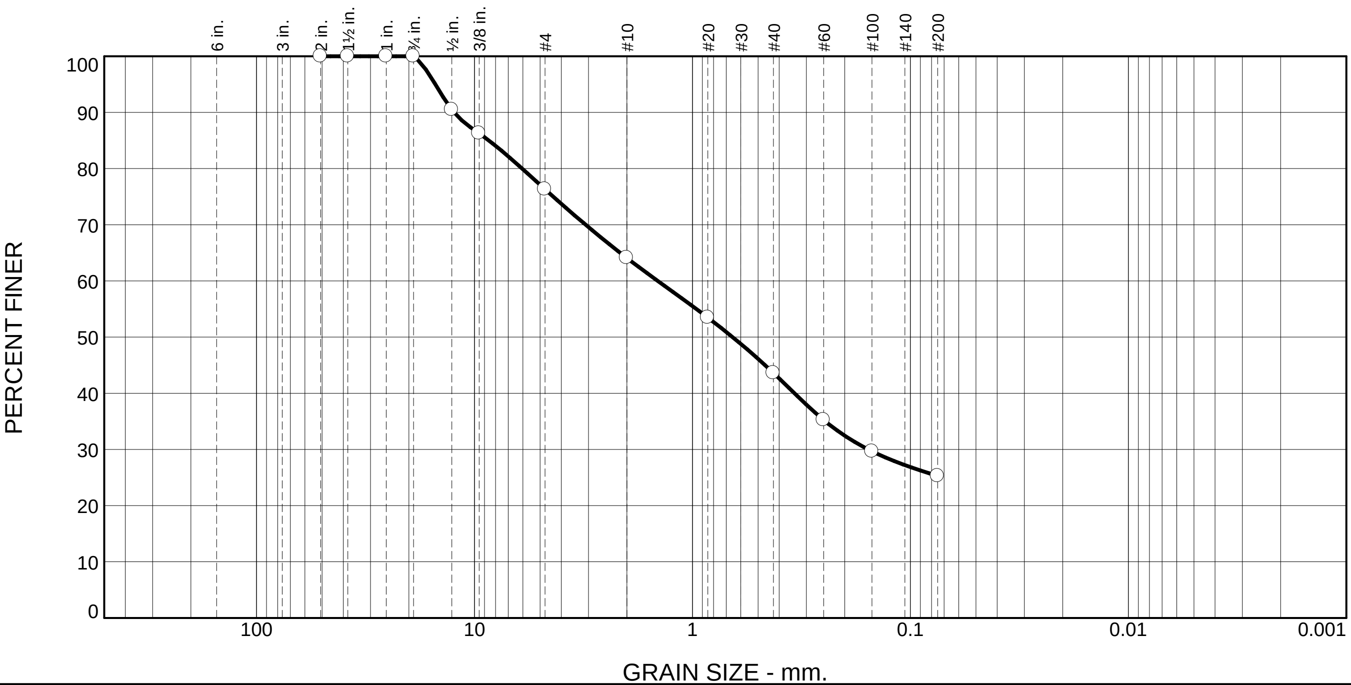
KIM ENGINEERING, INC.

Beltsville, Maryland

Client: Manna Woodward Studios, Inc.
Project: Coppin State University PSF Site A

Project No: G22091 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	23.7	12.2	20.5	18.3	25.3	

TEST RESULTS

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
2"	100.0		
1 1/2"	100.0		
1"	100.0		
3/4"	100.0		
1/2"	90.5		
3/8"	86.3		
#4	76.3		
#10	64.1		
#20	53.5		
#40	43.6		
#60	35.2		
#100	29.7		
#200	25.3		

* (no specification provided)

Material Description

silty sand with gravel (SM)

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= SM **AASHTO (M 145)=** A-1-b

Coefficients

D_{90}= 12.4090 D_{85}= 8.5986 D_{60}= 1.4428
D_{50}= 0.6545 D_{30}= 0.1561 D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 11/23/2022 **Date Tested:** 12/08/2022

Tested By: KS

Checked By: TL

Title: Principal Engineer

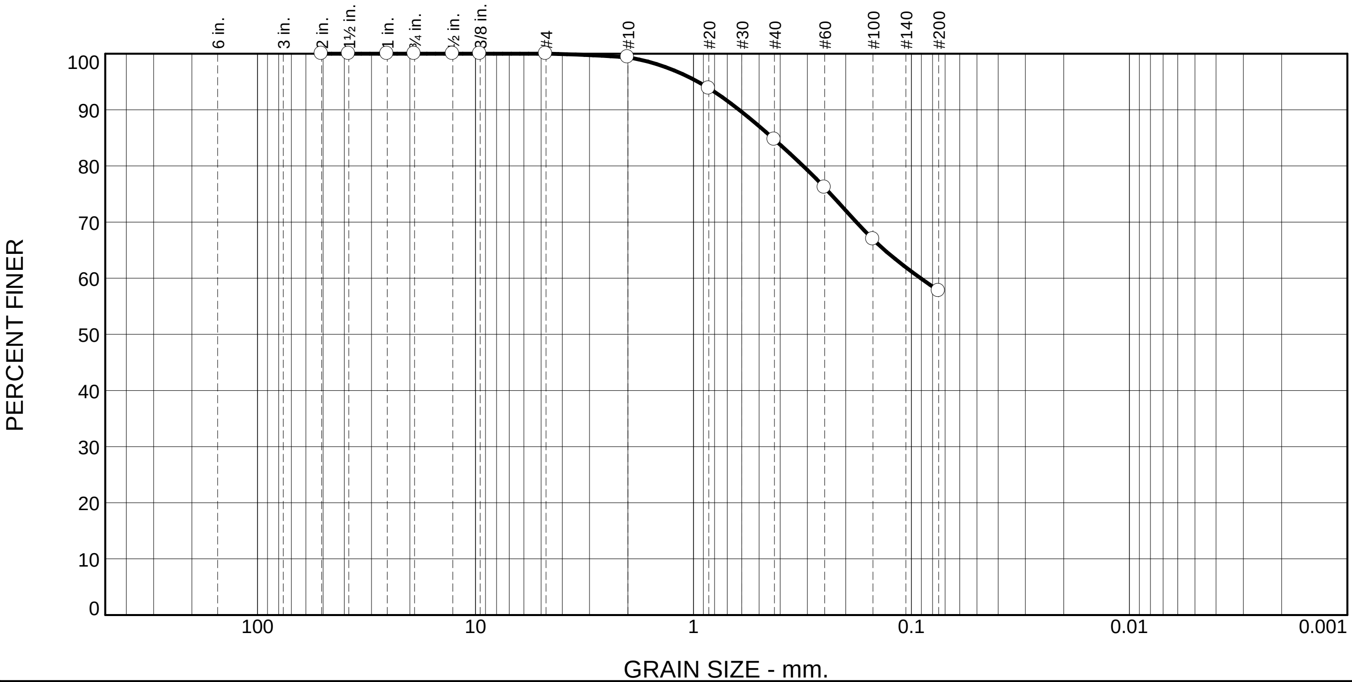
Source of Sample: B-2 **Depth:** 7'6"-9'00"
Sample Number: S-4
Date Sampled: 11/23/2022

KIM ENGINEERING, INC.

Beltsville, Maryland

Client: Manna Woodward Studios, Inc.
Project: Coppin State University PSF Site A
Project No: G22091 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.6	14.7	26.9	57.8	

TEST RESULTS			
Opening Size	**Percent Finer**	**Spec.* (Percent)**	**Pass? (X=Fail)**
2"	100.0		
1 1/2"	100.0		
1"	100.0		
3/4"	100.0		
1/2"	100.0		
3/8"	100.0		
#4	100.0		
#10	99.4		
#20	93.8		
#40	84.7		
#60	76.2		
#100	66.9		
#200	57.8		

* (no specification provided)

Material Description

sandy silt (ML)

Atterberg Limits (ASTM D 4318)

PL= 25 **LL=** 35 **PI=** 10

Classification

USCS (D 2487)= ML **AASHTO (M 145)=** A-4(4)

Coefficients

D_{90}= 0.6168 D_{85}= 0.4338 D_{60}= 0.0907
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 11/22/2022 **Date Tested:** 12/08/2022

Tested By: KS

Checked By: TL

Title: Principal Engineer

Source of Sample: B-4 **Depth:** 18'6"-20'00" **Date Sampled:** 11/22/2022
Sample Number: S-7

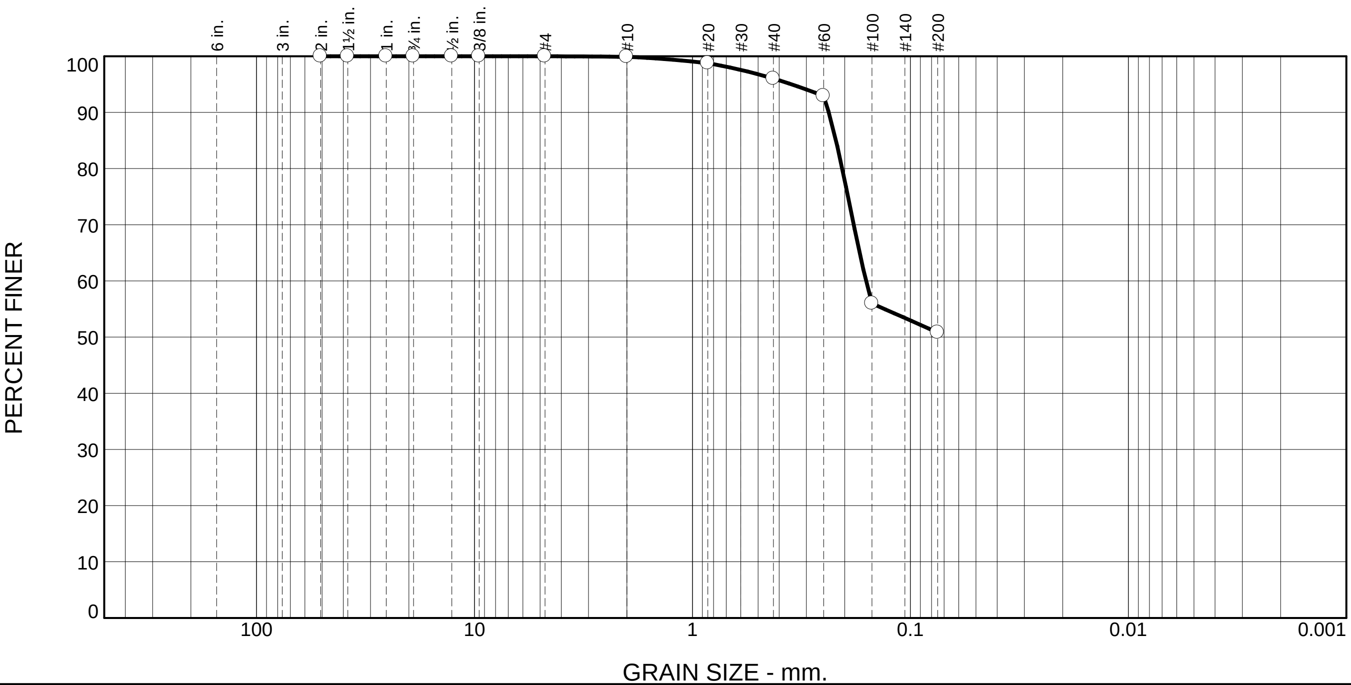
KIM ENGINEERING, INC.

Beltsville, Maryland

Client: Manna Woodward Studios, Inc.
Project: Coppin State University PSF Site A

Project No: G22091 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.1	3.9	45.2	50.8	

TEST RESULTS

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
2"	100.0		
1 1/2"	100.0		
1"	100.0		
3/4"	100.0		
1/2"	100.0		
3/8"	100.0		
#4	100.0		
#10	99.9		
#20	98.8		
#40	96.0		
#60	92.9		
#100	56.0		
#200	50.8		

* (no specification provided)

Material Description

sandy elastic silt (MH)

Atterberg Limits (ASTM D 4318)

PL= 30 **LL=** 52 **PI=** 22

Classification

USCS (D 2487)= MH **AASHTO (M 145)=** A-7-5(8)

Coefficients

$D_{90}=$ 0.2367 $D_{85}=$ 0.2195 $D_{60}=$ 0.1598
$D_{50}=$ $D_{30}=$ $D_{15}=$
$D_{10}=$ $C_u=$ $C_c=$

Remarks

Date Received: 11/23/2022 **Date Tested:** 12/08/2022

Tested By: KS

Checked By: TL

Title: Principal Engineer

Source of Sample: B-5 **Depth:** 18'6"-20'00" **Date Sampled:** 11/23/2022
Sample Number: S-7

KIM ENGINEERING, INC.

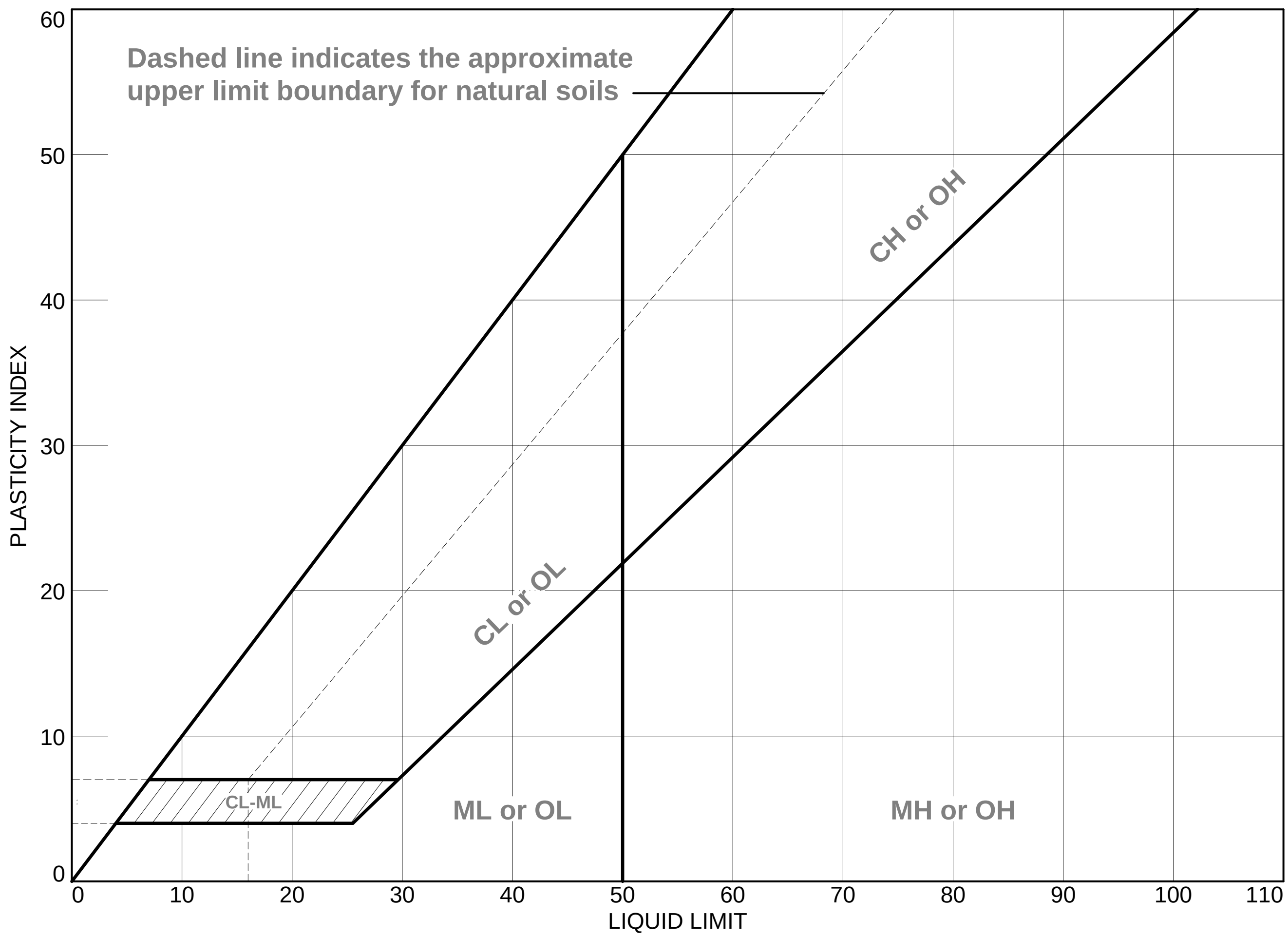
Beltsville, Maryland

Client: Manna Woodward Studios, Inc.
Project: Coppin State University PSF Site A

Project No: G22091 **Figure**

Liquid Limit and Plastic Limit Report

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	B-1	S-5	10'00"-11'6"	4.0	NP	NV	NP	SM

KIM ENGINEERING, INC.

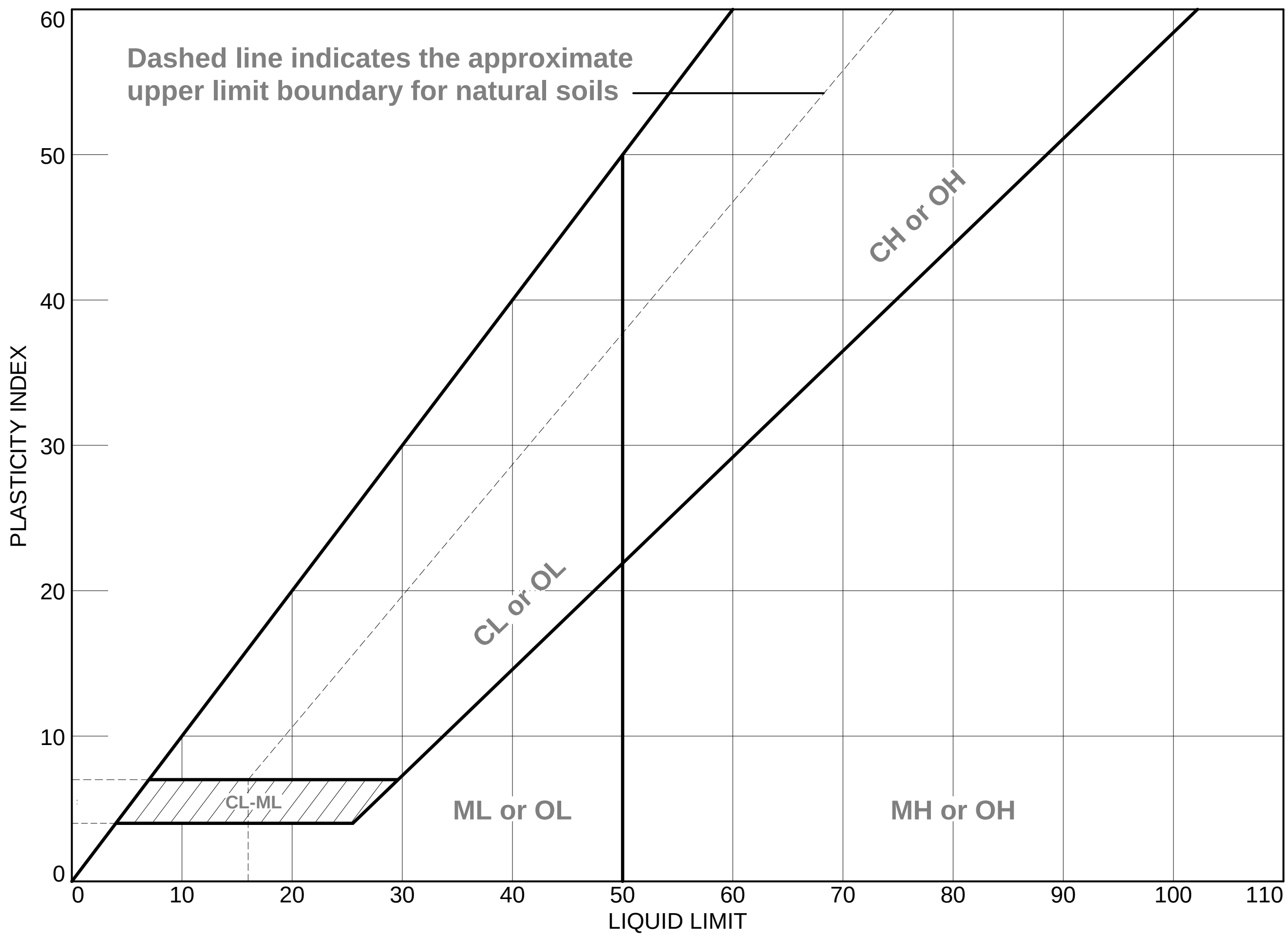
Beltsville, Maryland

Client: Manna Woodward Studios, Inc.
Project: Coppin State University PSF Site A

Project No.: G22091 **Figure**

Tested By: KS **Checked By:** TL

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	B-2	S-4	7'6"-9'00"	13.5	NP	NV	NP	SM

KIM ENGINEERING, INC.

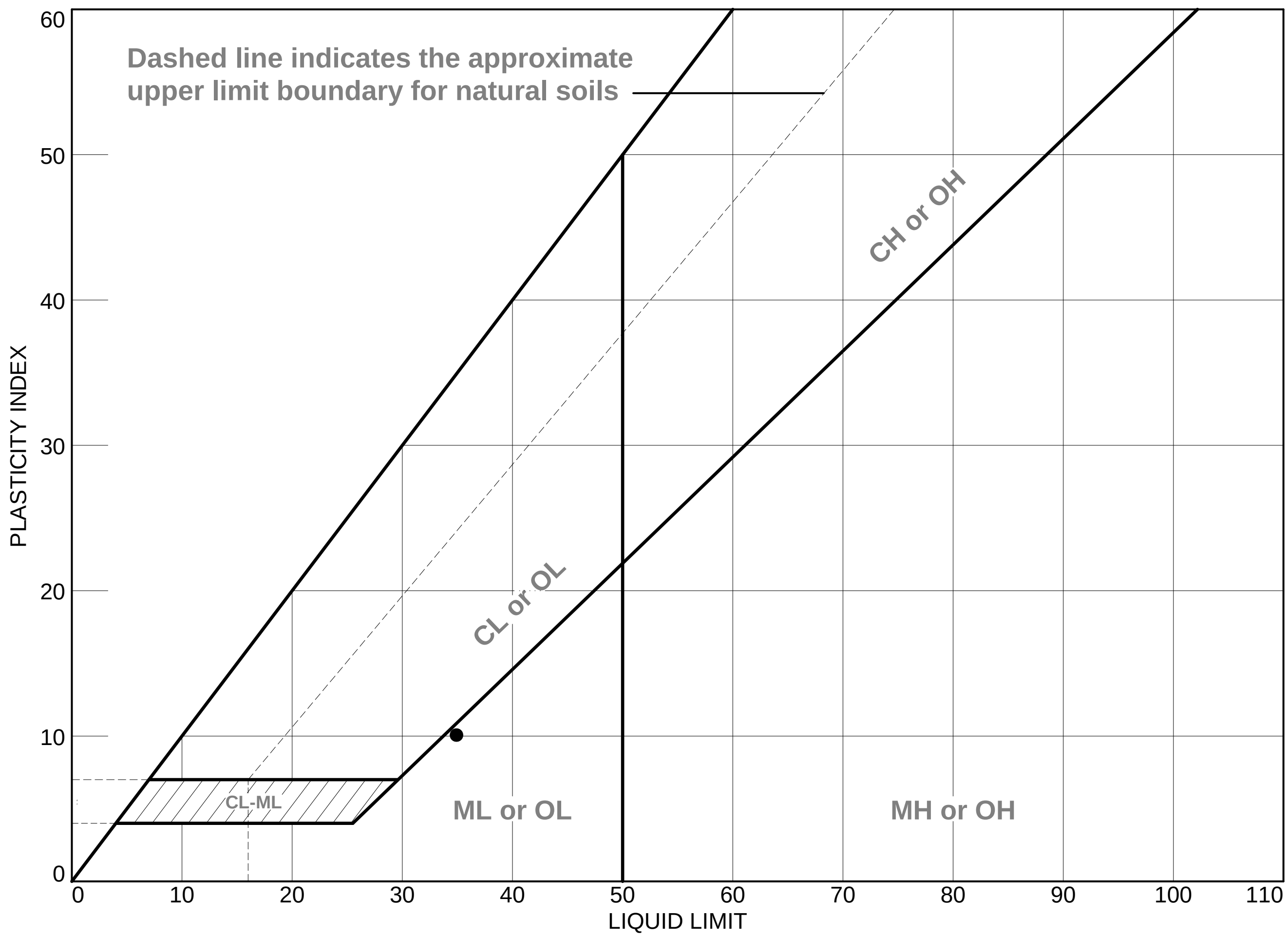
Beltsville, Maryland

Client: Manna Woodward Studios, Inc.
Project: Coppin State University PSF Site A

Project No.: G22091 **Figure**

Tested By: KS **Checked By:** TL

LIQUID AND PLASTIC LIMITS TEST REPORT

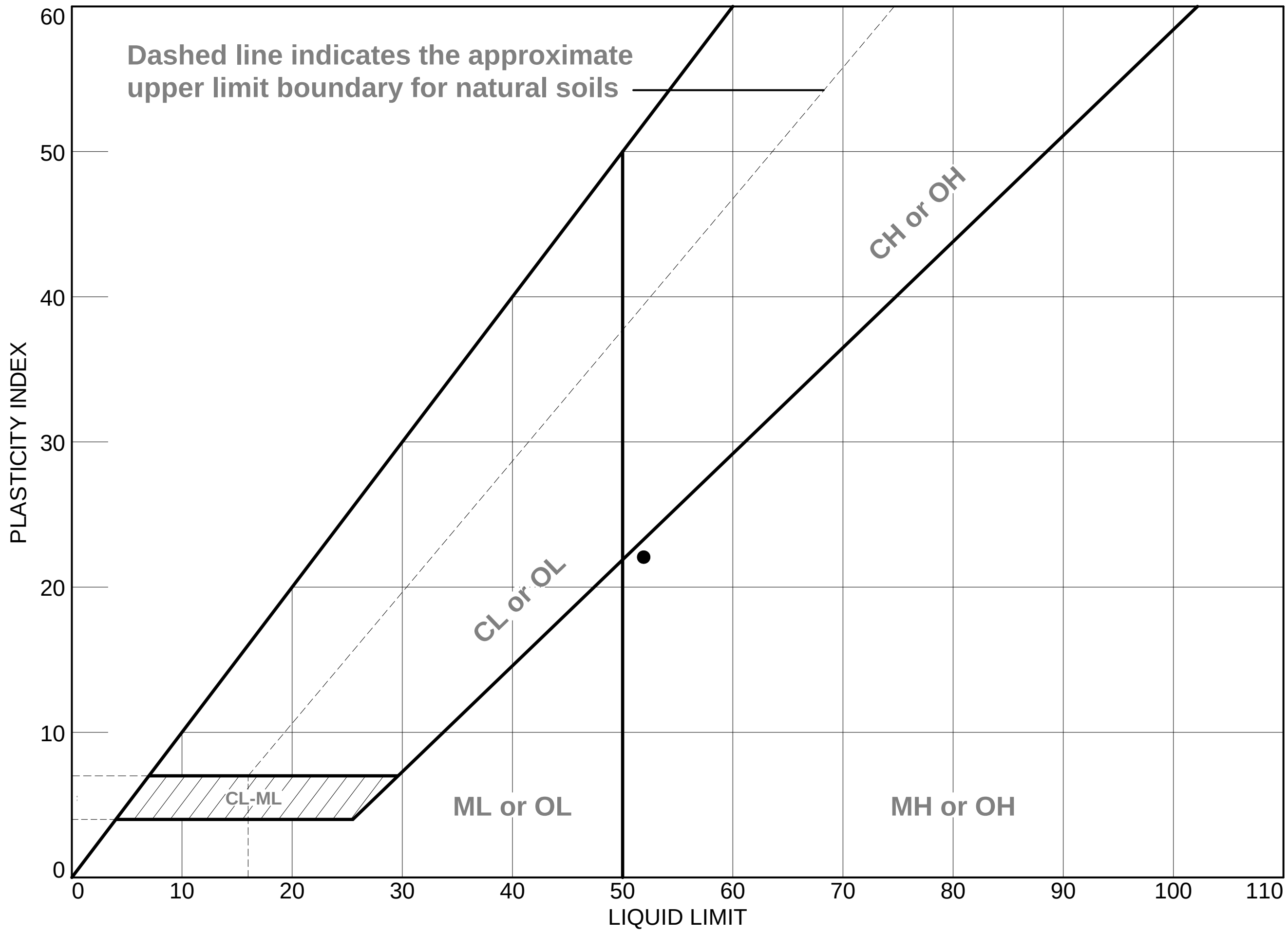


SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	B-4	S-7	18'6"-20'00"	11.8	25	35	10	ML

KIM ENGINEERING, INC. **Beltsville, Maryland**	**Client:** Manna Woodward Studios, Inc. **Project:** Coppin State University PSF Site A **Project No.:** G22091 **Figure**

Tested By: KS **Checked By:** TL

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	B-5	S-7	18'6"-20'00"	27.4	30	52	22	MH

KIM ENGINEERING, INC.

Beltsville, Maryland

Client: Manna Woodward Studios, Inc.
Project: Coppin State University PSF Site A

Project No.: G22091 **Figure**

Tested By: KS **Checked By:** TL

APPENDIX D

SEISMIC DESIGN PARAMETERS

Latitude, Longitude: 39.30674758, -76.65792019

Date	12/7/2022, 4:39:03 PM
Design Code Reference Document	ASCE7-16
Risk Category	III
Site Class	D - Default (See Section 11.4.3)

Type	Value	Description
S_S	0.141	MCE_R ground motion. (for 0.2 second period)
S_1	0.043	MCE_R ground motion. (for 1.0s period)
S_{MS}	0.225	Site-modified spectral acceleration value
S_{M1}	0.104	Site-modified spectral acceleration value
S_{DS}	0.15	Numeric seismic design value at 0.2 second SA
S_{D1}	0.069	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	B	Seismic design category
F_a	1.6	Site amplification factor at 0.2 second
F_v	2.4	Site amplification factor at 1.0 second
PGA	0.074	MCE_G peak ground acceleration
F_{PGA}	1.6	Site amplification factor at PGA
PGA_M	0.119	Site modified peak ground acceleration
T_L	6	Long-period transition period in seconds
SsRT	0.141	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	0.149	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.043	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.046	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.5	Factored deterministic acceleration value. (Peak Ground Acceleration)
PGA_{UH}	0.074	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.944	Mapped value of the risk coefficient at short periods
C_{R1}	0.928	Mapped value of the risk coefficient at a period of 1 s
C_V	0.7	Vertical coefficient

DISCLAIMER

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Latitude, Longitude: 39.30674758, -76.65792019

Date	12/9/2022, 3:01:34 PM
Design Code Reference Document	ASCE7-16
Risk Category	III
Site Class	C - Very Dense Soil and Soft Rock

Type	Value	Description
S_S	0.141	MCE_R ground motion. (for 0.2 second period)
S_1	0.043	MCE_R ground motion. (for 1.0s period)
S_{MS}	0.183	Site-modified spectral acceleration value
S_{M1}	0.065	Site-modified spectral acceleration value
S_{DS}	0.122	Numeric seismic design value at 0.2 second SA
S_{D1}	0.043	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	A	Seismic design category
F_a	1.3	Site amplification factor at 0.2 second
F_v	1.5	Site amplification factor at 1.0 second
PGA	0.074	MCE_G peak ground acceleration
F_{PGA}	1.3	Site amplification factor at PGA
PGA_M	0.097	Site modified peak ground acceleration
T_L	6	Long-period transition period in seconds
SsRT	0.141	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	0.149	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.043	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.046	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.5	Factored deterministic acceleration value. (Peak Ground Acceleration)
PGA_{UH}	0.074	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.944	Mapped value of the risk coefficient at short periods
C_{R1}	0.928	Mapped value of the risk coefficient at a period of 1 s
C_V	0.7	Vertical coefficient

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PRELIMINARY GEOTECHNICAL ENGINEERING REPORT

COPPIN STATE UNIVERSITY PUBLIC SAFETY FACILITY
SITE #2 (RAYNER AVENUE AND BRADDISH AVENUE)
BALTIMORE, MD
KIM PROJECT NO. G22091

PREPARED FOR
MANNS WOODWARD STUDIOS, INC.
10839 PHILADELPHIA ROAD
WHITE MARSH, MARYLAND 21162

PREPARED BY
KIM ENGINEERING, INC.
3916 VERO ROAD, SUITE K
BALTIMORE, MD 21127

KIM ENGINEERING

WWW.KIMENGINEERING.COM

December 14, 2022

Lindsey Kiefer, NCARB, AIA
Project Manager / Architect
Manns Woodward Studios, Inc.
10839 Philadelphia Road
White Marsh, Maryland 21162

Project: **Preliminary Geotechnical Engineering Investigation**
Coppin State University PSF (Site 2)
Baltimore, Maryland
KIM Project No. G21091

Dear Ms. Kiefer,

Kim Engineering Inc. (KIM) is pleased to submit a copy of our report for the above-referenced project. This investigation was conducted in accordance with our agreement dated August 31, 2022.

Services performed include five (5) SPT soil test borings, laboratory testing, and preparation of this preliminary geotechnical investigation report. Our geotechnical services report includes the following:

- An evaluation of the estimated subsurface soil conditions and groundwater conditions at the project site.
- Recommendations for different options of foundations and soil parameters for below-grade walls based on soil test borings and soil laboratory results.
- Seismic site classification information.
- Comments on geotechnical construction aspects that were readily apparent at the time of, in the area of, and to the depth of the investigation.

Services with respect to surveying for line and grade, specific dewatering recommendations, environmental matters, stormwater management recommendations, pavement section design, temporary slopes, seepage analysis, global slope stability analysis, erosion control, cost or quantity estimates, plans, specifications, and construction observation and testing were not included in the scope of services. Soil samples will be held for a period of thirty (30) days after the date of this report and then disposed of, unless an alternate disposition is requested.

We appreciate the opportunity to be of service to you for this project. If you have any questions regarding this project, please do not hesitate to contact either of the undersigned.

Very truly yours,
KIM ENGINEERING, INC.

Kamal Bhusal
Project Manager

Tom Labuda, PE, PG
Principal Engineer

PROFESSIONAL CERTIFICATION: I HEREBY CERTIFY THAT THESE DOCUMENTS WERE PREPARED OR APPROVED BY ME, AND THAT I AM A DULY LICENSED PROFESSIONAL ENGINEER UNDER THE LAWS OF THE STATE OF MARYLAND, LICENSE NO.:PE 42702 EXPIRATION DATE: 10-12-2024.

TABLE OF CONTENTS

1.0 SUMMARY OF CONCLUSIONS AND RECOMMENDATIONS 3
2.0 SITE DESCRIPTION AND PROPOSED CONSTRUCTION 4
3.0 SUBSURFACE EXPLORATION 4
3.1 Test Boring 4
4.0 GEOLOGY 6
5.0 SUBSURFACE CONDITIONS 6
5.1 General Stratification 6
5.2 Groundwater 8
6.0 SOIL GEOTECHNICAL LABORATORY TESTING 9
7.0 PRELIMINARY GEOTECHNICAL ENGINEERING ANALYSIS 10
7.1 Foundation Design Consideration 10
7.1.1 Conventional Spread Footings on Disintegrated Rock 11
7.1.2 Conventional Spread Footings on Impact Rammed Aggregate Piers 11
7.2 Ground Bearing Floor Slab 11
7.3 Seismic Site Coefficient 12
7.4 Below Grade Walls 13
7.5 Permanent Dewatering 14
7.6 Support of Excavation 15
8.0 CONSTRUCTION CONSIDERATIONS 15
8.1 General 15
8.2 Site and Subgrade Preparation 16
8.3 Excavation of Rock 16
8.4 Fill Material and Compaction 17
8.5 Groundwater Control and Site Drainage 17
8.6 Inspection of Subgrades 17
9.0 LIMITATIONS 18

APPENDIX A

Site Location Plan
Boring Location Plan

APPENDIX B

Subsurface Investigation
- Identification of Soil
- Record of Soil Exploration Logs

APPENDIX C

Geotechnical Laboratory Test Results
- Particle Size Distribution
- Liquid Limit and Plastic Limit Reports

APPENDIX D

Seismic Design Parameters

1.0 SUMMARY OF CONCLUSIONS AND RECOMMENDATIONS

The following is a summary of our conclusions and recommendations:

a Subsurface conditions in the proposed construction area generally indicate existing fill consisting of Silty Sand, sandy Silt, Gravel with debris in stratum A, residual soil consisting of silty SAND and sandy SILT (ML) in stratum B underlain by Decomposed Rock in stratum C.

b Recommended foundation options are presented in section 7.1 of this report. Feasibility of construction will be defined by presence of fill and disintegrated rock underlying the site.

c Compacted fill in structural areas should be classified as silty SAND (SM) or more granular per ASTM D 2487 and compacted to at least 95 percent of maximum dry density per ASTM D 698.

Variations in soil conditions may be encountered during construction. Determination of such variations will permit correlation between the subsurface exploration data of this report and actual conditions encountered during construction and verification of conformance with the plans and specifications. We recommend that Kim Engineering, Inc. be retained to perform professional observations of foundation subgrades.

This report is based on information provided to us on the proposed construction. If the project characteristics are changed from those indicated herein, our recommendations may require modifications. Please advise us of any changes in the proposed construction.

We recommend that the project specifications include the following statement:

> "A preliminary geotechnical report has been prepared for this project by Kim Engineering, Inc. and is available to prospective bidders and/or contractors for informational purposes only. The report has been prepared for design purposes only and may not be sufficient to prepare an accurate bid for construction. Contractors wishing copies of this report may secure them from Kim Engineering Inc. at a nominal charge with the understanding that its scope is limited solely to generalized design considerations."

We have prepared this report in accordance with contemporary geotechnical engineering practices and make no warranties, either expressed or implied, as to the professional services provided under the terms of our agreement and included in this report.

2.0 SITE DESCRIPTION AND PROPOSED CONSTRUCTION

The site is located in the Mosher district of Baltimore, Maryland, and is framed by Ashburton Street to the West, Rayner Avenue to the South, Braddish Avenue to the East, Jordan Street and West Lafayette Avenue to the North. The site consists of an open grass covered space with remnants of dilapidated asphalt parking pavement. The provided site plan indicates that the site's topography is slightly sloped from a high of approximately El. 168 ft in the southwest corner to approximately El. 140 ft in the southeast section of the site. The surface runoff is in a general west to east direction.

According to Google Earth historic images the site was occupied by Lutheran Hospital building, paved roadway, and paved parking area before the year 2007. The buildings were demolished, and the site was cleared between 2007 and 2008.

Based on the schematic site plan and information provided to us, the proposed construction will consist of a new one to five above ground levels and one to two below grade levels buildings, and associated facilities. The project is in the preliminary design stage and detail building plans and structural loads were not provided at the time of writing this report. We understand that the purpose of this subsurface investigation is to determine the feasibility of the site for planned development.

The entire fieldwork was done in readily accessible areas within the proposed construction area as per the boring location provided by the client. The site location plan is appended in Drawing No. 1 in Appendix A.

3.0 SUBSURFACE EXPLORATION

3.1 Test Boring

In order to evaluate the subsurface conditions of the site for the study, a total of five (5) standard penetration tests (SPT) borings (B-1 to B-5) were drilled at the site. The approximate location of the test borings is depicted on the attached Drawing No. 2 in Appendix A (Boring Location Plan).

The standard penetration tests borings were originally planned to extend to 35 feet and 50 feet. All borings were terminated above the planned depths on refusal. The table below summarizes the test boring schedule.

Table 1: Summary of Test Borings

Boring No.	Depth of Boring (ft)	Approximate Existing Ground Elevation (ft) (per site plan)	Depth to Disintegrated Rock (N>60bpf) (ft)
B-1	33.7	155	28.5
B-2	33.6	147	33.5
B-3	33.7	144	28.5
B-4	33.6	156	10
B-5	33.8	140	28.5

The test borings were accomplished using a track mounted drill rig CME-55. The exploration program was performed in the field on November 29th to December 1st, 2022. Hollow-stem augers were advanced to pre-selected depths and representative soil samples were recovered with a standard split-spoon sampler in general accordance with ASTM D-1586. Disturbed representative soil samples were recovered while performing the Standard Penetration Test (SPT). This test (ASTM D-1586) consists of a 140-pound (lb) hammer falling 30 inches. The number of blows required to drive the standard split spoon sampler (2-inch O.D., 1-3/8-inch I.D.) a distance of 12 inches after an initial set of 6 inches to ensure the sampler is in undisturbed material, is recorded as the Standard Penetration Resistance (N-value) of the soil.

The N-value, for the majority of subsurface situations, provides a generalized indication of in-situ soil conditions when reviewed by individuals with established geotechnical backgrounds. N-values can be used to provide a qualitative indication of the in-place relative density of granular soils. Similarly, N-values provide an indication of consistency for cohesive soils.

Subsurface water level readings were taken in each of the test borings during drilling, at the completion of the drilling process in all soil borings and 24 hours after the drilling process in the soil borings B-3 and B-4. Upon completion, the boreholes were backfilled with auger cuttings (spoils) and grout on top 10 feet. The backfill material was compacted to the extent feasible; however, some subsidence of the backfill could occur at a future date. As a result, it is recommended that the boreholes be monitored periodically.

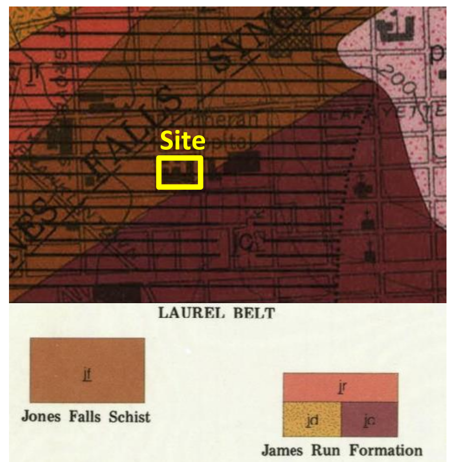
Representative portions of the split-spoon soil samples obtained throughout the exploration program were placed in glass jars and transported to our laboratory for further evaluation and visual classification per the visual-manual identification procedure (ASTM D-2488) and the Unified Soil Classification System. The soil descriptions and classifications discussed in this report and shown on the attached boring logs are based on visual observation and as previously noted, should be considered approximate.

Soil samples recovered on this project will be stored at Kim Engineering, Inc. for a period of thirty (30) days from the date of this report. After thirty (30) days, the samples will be discarded unless prior notification for an alternate disposition is provided to us in writing.

4.0 GEOLOGY

According to the "*Geological Map of the Baltimore West Quadrangle, Maryland*" by William P. Crowley and Juergen Reinhardt (1979), the site is underlain mainly by Jones Falls Schist and described as:

"Medium- to coarse-grained biotite-plagioclase-muscovite-quartz schist, in places accompanied by fine-grained biotite-plagioclase-quartz gneiss in layers a few centimeters thick. Garnet, and less commonly tourmaline, occur in some outcrops. Includes very minor muscovite-plagioclase-quartz schist, quartzite, amphibolite, and muscovite-quartz-feldspar gneiss.



The southeastern portion of the site is underlain by Carroll Gneiss Member (jc) of James Run Formation.

5.0 SUBSURFACE CONDITIONS

5.1 General Stratification

The subsurface conditions discussed below and those shown on the boring logs represent an estimate of the subsurface conditions based on an interpretation of the boring data using geotechnical engineering judgment. Transitions between different soil strata are usually less distinct than those shown on the boring logs. Although individual test borings are representative of the subsurface conditions at the boring locations on the dates shown, they are not necessarily indicative of subsurface conditions at other locations or at other times.

More comprehensive descriptions of the materials encountered are included in the attached test boring logs. The subsurface investigation indicated that the following generalized strata underlie the site in the areas and to the depths investigated.

Ground Cover:
Borings indicated four (4) to six (6) inches of topsoil beneath the ground surface.

Stratum A (Existing Fill):
Existing Fill was encountered below the ground cover at all the test boring locations. The fill material ranges from the depth of 10 feet to 28.5 feet. The encountered fill generally consisted of silty Sand with mica, gravel, asphalt, glass, wood, brick, and concrete fragments. The Standard Penetration Test (SPT) N-values in the existing fill ranged from 2 blows per foot (bpf) to 50 blows per 2 inches. The depth for the existing undocumented fill is presented in the following table.

Table 2: Summary of Existing Fill

Boring Identification	Depth of Fill (ft)	Existing Fill Bottom Elevation (ft)
B-1	23.5	131.5
B-2	28.5	118.5
B-3	28.5	115.5
B-4	10	146
B-5	23.5	23.5

Stratum B (Residual Soil)
The natural residual soils were encountered below the existing fill at the test boring locations B-1, B-2, B-3, and B-5. The soil generally consisted of silty SAND (SM) and sandy SILT (ML). The SPT N-values obtained in the coarse-grained soil ranged from 3 to 37 bpf, indicating very loose to dense relative density. The SPT N-values obtained in the fine-grained soils ranged from 4 to 6 bpf, indicating soft to medium stiff consistency.

Stratum C (Disintegrated Rock)
Decomposed rock (Disintegrated Rock), identified as residual material with an N-value greater than 60 bpf, was encountered at various depths across all soil borings. This stratum was identified beneath the stratum A and Stratum B in all the boring locations. The depth to the decomposed/disintegrated rock is provided in table 1.

Disintegrated Rock (also known as decomposed rock) is defined as a residual material with a penetration resistance (N-value) of more than 60 blows per foot and less than refusal (50 blows per 2-inch penetration). It typically retains the remnant rock structure of the parent rock (i.e., is saprolitic) but exhibits the engineering characteristics of a soil when removed. Within a disintegrated rock zone, it is not uncommon to encounter slabs of rock, rock lenses, and/or boulders of intact rock. Also, disintegrated rock levels can vary significantly throughout a particular project site.

It must be stressed that the composition of the disintegrated rock material described on the test boring logs is based on a visual observation of material removed with the auger. In situ materials are very dense rock-like to rock materials. Excavation difficulty as well as specialized excavation techniques should be anticipated in the decomposed rock materials especially in the denser and/or deeper portions of the media.

All borings encountered spoon refusal defined as an N-value of 50 over 2 inches or less. This could be an indication of the top of the Bedrock at these locations. Rock coring was not part of our scope of work.

The soil symbols indicated in the stratum descriptions and on the boring logs represent the Unified Soil Classification (ASTM D-2488) group symbols and are based primarily on visual observation of the specimens recovered. Criteria for visual-manual classification of soil samples are given in Appendix B of this report.

5.2 Groundwater

Groundwater observations were performed at the test boring locations. Groundwater was recorded during drilling or at completion of the drilling and 24 hours after the drilling operation. Groundwater was encountered at the depth ranging 14.3 feet to 29 feet. The depth for the observed groundwater is presented in the following table.

Table 3: Summary of Groundwater

Boring Identification	Groundwater Readings	
	During Drilling / End of Drilling (ft)	24 Hours After Completion of Drilling (ft)
B-1	18.5/14.3	Not measured
B-2	23.5/NA	Not measured
B-3	23.5/19.0	19
B-4	33.5/29	28
B-5	18.5/NA	Not measured

Groundwater level readings are considered to be reliable indication of the water levels at the time indicated. However, fluctuations of groundwater levels as well as perched water may be expected with variations in precipitation, evaporation, surface runoff, and related factors.

6.0 SOIL GEOTECHNICAL LABORATORY TESTING

Geotechnical laboratory testing was performed on jar samples obtained from selected test borings for soil classification, plasticity index, moisture content and standard proctor. Tests were performed in accordance with their associated ASTM Standards. The test results are presented in Appendix C. The associated ASTM methods are presented below:

ASTM Method	Description
D-2216	Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
D-422	Standard Test Method for Particle-Analysis (Grain Size)
D-4318	Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Table 4: Summary of Laboratory Test Results:

Boring Identification	Sample No.	Depths (ft)	Percent Fines (-#200)	Liquid Limit (LL)	Plasticity Index (PI)	Natural Moisture (%)	USCS
B-1	S-5	10.0-11.5	75.8	30	13	17.6	CL
B-2	S-9	28.5-30.0	55	NV	NP	33.8	ML
B-3	S-8	23.5-25.0	58.4	NV	NP	39.2	ML
B-5	S-8	23.5-25.0	47.8	NV	NP	23.4	SM

USCS Soil classification as determined by the Unified Soil Classification System.

LL: Liquid limit: the moisture percentage at which soil behavior transitions from plastic to liquid.

PI: Plastic index: The difference between the plastic and liquid limits (PI = LL – PL), indicates the range of moisture that the soil acts in a plastic manner. The plastic limit is defined as the minimum moisture percentage at which a soil behaves in a plastic manner.

NP Non-Plastic.

NV Non-Viscous

7.0 PRELIMINARY GEOTECHNICAL ENGINEERING ANALYSIS

The following evaluations and recommendations are based on our observations at the site, interpretation of the field data obtained during this exploration, and our experience with similar subsurface conditions and projects. Soil penetration data have been used to estimate an allowable bearing pressure using established correlations. Subsurface conditions in unexplored locations may vary from those encountered.

Determination of an appropriate foundation system for a given structure is dependent on the proposed structural loads, soil conditions, and construction constraints such as proximity to other structures, etc. The subsurface exploration aids the geotechnical engineer in determining the soil stratum appropriate for structural support. This determination includes considerations with regard to both allowable bearing pressure and compressibility of the soil strata. In addition, since the method of construction greatly affects the soils intended for structural support, consideration must be given to the implementation of suitable methods of site preparation, fill compaction and other aspects of construction. The following foundation design criteria are preliminary and provided for planning purposes only. Once the architectural and structural designs are finalized, KIM should review copies of the plans and specifications to revise or expand our recommendations.

7.1 Foundation Design Consideration

Soil profiles encountered across the proposed new construction site were defined by uncontrolled fill up to depths of approximately 28.5 feet consisting of loose to very dense silty Sand, sandy Silt, sandy lean Clay, Gravel with various amounts of deleterious and organic matter. We understand that the proposed structure will be up to 5 stories high and with up to 2 underground levels. The lowest level floor elevation is planned at approximately (±) 20 feet below the existing ground elevation.

Based on the results of the field subsurface investigation deep fill and depth to disintegrated rock or bedrock will govern the foundation design. The existing fill is not suitable to support the new building. Conversely, the relatively shallow disintegrated rock may require rock excavation methods for the 2-levels below grade parking garage and basement planned for this project. Verification rock coring was not included in our scope of work.

Based on or subsurface exploration and our experience with similar subsurface conditions and projects, the following foundation options are proposed for the design.

7.1.1 Conventional Spread Footings on Disintegrated Rock

The existing disintegrated rock or dense natural soil encountered during this exploration are considered suitable for support spread footings. The foundations should be proportioned for a net allowable soil bearing pressure of 6,000 psf when founded on approved natural granular soils of Stratum B or on decomposed rock (N>60 bpf) and net allowable soil bearing pressure of 10,000 psf when founded on competent Bedrock. The depth to and quality of the bedrock should be verified by additional geotechnical investigation. We do not recommend placing new foundations on the existing fill.

7.1.2 Conventional Spread Footings on Impact Rammed Aggregate Piers

The use of Impact system Rammed Aggregate Piers (RAP), at least 24 inches in diameter are an alternative method to improve the foundation subgrade soils consisting of unsuitable soil and fill. The RAP piers should penetrate through the existing fill and terminate in dense disintegrated rock or on top of the bedrock below.

7.2 Ground Bearing Floor Slab

We do not recommend supporting the concrete slab-on-grade on existing fill. The presence of soft, loose, and organic compressive matter in fill will cause differential settlement and damage to the concrete surface. If the deep fill cannot be safely excavated and replaced with new compacted fill, the structurally supported slab will be required in this location.

For slabs placed on new compacted structural fill we recommend a modulus of subgrade reaction (k) of 125 pounds per cubic inch (pci) for approved subgrades (k value considers a 1-ft by 1-ft square plate). A minimum 6-inch-thick layer of free-draining aggregate is recommended to be placed below the floor slab to serve as a capillary moisture barrier. A polyethylene membrane or similar vapor barrier should be placed over the aggregate to prevent concrete contamination. Proper mix designs, placement methods, and curing methods must be utilized to reduce the potential for concrete shrinkage issues and curling that are sometimes associated with the use of a vapor barrier. Control joints should be provided to control shrinkage cracks of the concrete floor system.

Slab subgrades are often disturbed after final grading due to ongoing construction activities, utility installations, and weather conditions. We recommend that subgrades that become saturated or lose their support capabilities be removed and replaced with new selected compacted engineered fill.

7.3 Seismic Site Coefficient

We are providing a Seismic Site Class Definition per the 2018 International Building Code (IBC) and American Society of Civil Engineers ASCE 7 guidance.

Our scope of services did not include a seismic conditions survey to determine site-specific (accurate) shear wave velocity information. IBC 2018 provides a methodology for interpretation of Standard Penetration Test resistance values (N-values) to determine a Site Class Definition. However, this method requires averaging N- values over the top 100 feet of the subsurface profile, a depth well in excess of the depths of the test borings.

Based on the subsurface data presently obtained and in general accordance with the 2018 IBC, it appears reasonable to assign the site a Classification "D". However, lowering the building foundations to bedrock will allow for higher Classification "C" in design.

The "U.S. Seismic Design Map Web Application" available through the USGS and ASCE websites provides hazard curves, uniform hazard response spectra, and design parameters. These parameters were developed using two percent probability of exceedance (PE) in 50 years. The mapped spectral response acceleration values for the project site are provided in the table below.

Table 5: Mapped Spectral Response Acceleration Values (Class C and D)

Description	Period (Sec)	Sa
Mapped Short Period Spectral Response Acceleration (S_s)	0.2	0.14
Mapped 1-Second Period Spectral Response Acceleration (S_1)	1.0	0.043

For a Site Class C and D, with the above-indicated mapped spectral acceleration values and risk category III, the calculated site coefficient values and the maximum and design spectral response acceleration values are provided in the table below.

Table 6: Site Coefficients, and Design Spectral Response Acceleration (Class C and D)

Soil and Rock Profile	Soil	Dense Soil and Rock
Seismic Site Class	D	C
Site Coefficient (F_a)	1.6	1.3
Site Coefficient (F_v)	2.4	1.5

Short Period, Maximum Spectral Response Acceleration (S_{MS})	0.225	0.183
1.0 Second Period, Maximum Spectral Response Acceleration (S_{M1})	0.103	0.065
Short Period, Design Spectral Response Acceleration (S_{DS})	0.15	0.122
1.0 Second Period, Design Spectral Response Acceleration (S_{D1})	0.069	0.043

Based on our subsurface investigation and engineering judgement, the proposed site is not susceptible to liquefaction under the design earthquake magnitude provided by the code.

7.4 Below Grade Walls

Below-grade walls or basement walls associated with the project should be designed to withstand lateral earth pressures from the backfill and supported soils. Additionally, the walls should be designed to resist the lateral components of surcharge loads occurring within a zone defined by a plane extending up at a 45-degree angle from the base of the wall.

We recommend that the buildup of hydrostatic pressures be precluded by specifying a free-draining fill material immediately adjacent to below-grade walls, with a gravity-driven subdrainage system at the base of the walls.

Earth pressures on walls below grade are influenced by structural design of the walls, conditions of wall restraint, methods of construction and/or compaction and characteristics of the materials being restrained. The most common conditions assumed for earth retaining wall design are the active and at-rest conditions. Active conditions apply to relatively flexible earth retention structures, such as free-standing walls that require rotation and movement to mobilize soil shear strength without affecting their function. Basement walls are rigidly constrained and should be designed utilizing at-rest conditions. A passive condition also exists to represent the maximum possible pressure that may be developed by soils resisting the forces exerted by the active or at-rest conditions. The magnitude of movement required to completely mobilize the passive forces is often beyond aesthetic and/or structural design tolerances in addition to uncertainties during foundation construction, use of passive pressure should be used cautiously, if at all, and be assigned a factor of safety of no less than two (FS>2).

To prevent unforeseen increases in lateral loading, large vehicular and heavy excavation equipment should not operate within a lateral distance equal to the wall height or five (5) feet, whichever is greater. Grading during site development and construction should be maintained to meet the intent of the final design, thus preventing channeled drainage toward partially complete retaining wall structures that could result in delay or damage. This may require diversion dikes, level spreaders, or berms that are not depicted on the erosion and sediment control plan. It is highly recommended that these changes be discussed with the civil design firm to verify that they will not overload storm water management facilities.

The underlying table provides typical parameters for AASHTO #57 crushed stone as well as the encountered on-site soils and import material that might be utilized for the design of retaining structures/walls. The values assigned to the latter are somewhat conservative due to the variable composition of representative samples. Suitable on-site soils would include silty sand (SM) after verification of natural moisture content.

Table 7: Below-Grade Wall Design Parameters

Earth Pressure Condition	AASHTO #57	On-Site Soils [1]
Active (K_A)	0.22	0.33
At-Rest (K_O)	0.36	0.5
Passive (K_P)	4.60	3.0
Moist Unit Weight (γ)	110 pcf	120 pcf
Angle of Internal Friction (ϕ)	40°	30°
Sliding Coefficient (soil-concrete)	0.55	0.35

Note 1: Classified as Silty Sand ("SM"), or better

Use of the parameters assumes that a full-height drainage system has been installed and maintained during construction and throughout the life of the structure. The system should conform to section 1805 of the IBC relating to damp-proofing as groundwater is in excess of five (5) feet below the potential foundation elevation assumed for the project.

7.5 Permanent Dewatering

If the subbase level is not designed for hydrostatic pressure, a permanent dewatering system should be implemented to prevent the groundwater from impacting the structure and to minimize the transmission of moisture through walls. To avoid producing hydrostatic pressures on the sublevel walls, it is recommended that an approved vertical drain be constructed along the entire exterior of the below grade walls. The system would incorporate drain tile in Maryland No. 57 stone enveloped with filter fabric to route the water to sumps and sump pumps.

It is recommended that a subfloor drainage (subdrainage) system be installed below the concrete floor slab of any underground spaces to preclude development of hydrostatic uplift pressure on the lowest level floor slab and to promote a dry space. A subdrainage system consisting of perforated pipe placed in gravel-filled trenches may be installed beneath the slab on grade to control groundwater. Gravel should be wrapped in non-woven drainage filter fabric. The perimeter line may be installed running around the interior perimeters of basement areas with an adequate slope to facilitate efficient water removal and be designed to discharge to sump pit and pump systems. Interior subfloor drainage system and exterior drainage system could be connected with weep holes and or bleeder pipes in order to make flow of water to the sump pit and pump system.

7.6 Support of Excavation

It is anticipated that temporary excavation support will be required during construction. In our opinion, excavation support consisting of soldier piles and timber lagging is considered suitable. Due to the depth of the excavation, soldier piles system with tie backs is appropriate. A soldier pile wall consists of vertical steel beams, typically spaced from 6 to 10 feet apart along the proposed excavation wall, spanned by timber lagging. Prior to the excavation, the steel beams are installed to the designed depth and then backfilled with concrete. Timber lagging is installed between the piles to further stabilize the walls of the excavation. The excavation support should be designed to resist the full earth, water, and surcharge loads acting on it. Surcharge loads from the construction equipment's must be considered. Other additional loads may be required based on the Contractor's planned construction methods.

8.0 CONSTRUCTION CONSIDERATIONS

8.1 General

The principal purpose of this section is to comment in general on the items related to foundation construction, earthwork, and related geotechnical engineering aspects of construction that should be expected for this project. It is recommended that the geotechnical engineer be retained to provide soil engineering services during the actual site preparation and foundation construction phases of the project to perform appropriate evaluations to help ensure that conditions encountered during construction are similar to conditions encountered in the borings. The geotechnical engineer can also assist in interpretation of differing subsurface conditions that may be encountered and recommend remedial work, if needed.

8.2 Site and Subgrade Preparation

Areas proposed for grading or construction should be stripped and grubbed of all existing pavement, topsoil, vegetation, roots, organics, and loose and soft on-site soils before placing structural fill. In addition, existing foundations, abandoned utilities, underground tanks, cisterns, or surface drainage systems such as field tile or perforated pipes possibly encountered in the construction areas should be undercut, removed, or appropriately plugged and backfilled with structural fill in accordance with the recommendations provided in Section 8.4 of this report and at the discretion of a Geotechnical Engineer.

Following preparation of exposed subgrades, accessible portions of the new structure and pavement subgrade should be proof rolled with a loaded 20-ton tandem axle dump truck and witnessed by the Geotechnical Engineer or qualified representative. The purpose of the proof rolling will be to locate any isolated soft, unstable or "pumping" pockets of soil, which should be excavated or otherwise stabilized as directed by the Geotechnical Engineer. Proper site drainage should be maintained at all times to prevent ponding of water at the site during construction. If the soils do become wet, care should be taken to minimize heavy construction equipment from operating on the prone subgrade.

The temporary grades should be sloped at no steeper than 1:5 horizontal to 1 vertical (1.5H:1V). All cleared and grubbed material should be disposed of outside and below the limits of the project area.

8.3 Excavation of Rock

Rock coring was not performed in the site. However, bedrock is expected across the project site according to the borings SPT N number. Additional soil borings with rock coring is recommended in order to verify the depth to the rock and rock quality. Heavy duty excavation equipment such as backhoes equipped with rock teeth or bulldozers equipped with ripping attachments can sometimes excavate highly weathered bedrock. However, blasting could be required, if the bedrock is above the basement levels of the proposed building.

For weathered and highly fractured bedrock, there is some potential for localized instability. In such cases, careful inspection during construction and installation of a shoring system is recommended.

8.4 Fill Material and Compaction

The project near-surface soils generally consisted of existing fill consisting of silty Sand, sandy Silt and naturally occurring soils consisting of sandy SILT (ML) and silty SAND (SM). On-site soil i.e., silty SAND (SM) that is free of organic matter or debris, waste materials, frozen materials is considered to be suitable for reuse as compacted engineered fill. Sorting to remove existing fill material and oversized material (larger than 3 inches in diameter) may be required. Proposed fill material that will be subject to third party compaction testing should be subjected to laboratory analysis consisting of, but not necessarily limited to, Proctor moisture/density determination, Atterberg limits, and gradation.

If imported fill is required at the site, we recommend that the material have low expansive characteristics and shall have Unified Soils Classification (ASTM D 2487) of ML or better. Any imported soil fill required to balance the site should adhere to the following parameters unless specifically accepted in writing by the Geotechnical Engineer at time of placement:

Maximum Dry Density (ASTM D698)	$\geq$ 110 pcf
Liquid Limit	$\leq$ 30
Plasticity Index	$\leq$ 15
Expansion Index	$\leq$ 40

We recommend that the fill material be placed in lifts having a maximum loose lift thickness commensurate with the equipment being utilized to perform the compaction. In no case should those lifts exceed eight (8) inches. Each lift should be uniformly compacted to at least 95% of the laboratory maximum dry density as determined by ASTM D 698 based on Baltimore City requirements.

8.5 Groundwater Control and Site Drainage

Based upon the borings, groundwater will be encountered during construction. Installation of a perimeter construction dewatering system may be required for deep excavation. The system selection, design, and testing should be provided by a specialty dewatering contractor with local practice of at least 5 years.

8.6 Inspection of Subgrades

We recommend that all subgrades be inspected by a Geotechnical Engineer or an experienced engineering technician. Subgrades should be tested to check whether any unstable areas exist.

Any unstable zones that are identified that cannot be re-compacted should be undercut to a depth, within the area marked by the inspecting engineer. The depths and extent of undercuts should be determined by the inspecting Geotechnical Engineer. Deeper undercuts should be avoided, and it is requested that KIM be extended an opportunity to review the conditions warranting any deeper undercuts before undercutting commences. Undercut volume should be backfilled to grade with compacted fill in accordance with the requirements in this report.

9.0 LIMITATIONS

This report has been prepared for the exclusive use by our client for specific application to the proposed construction as presented herein. Our services were performed in accordance with contemporary soil and foundation engineering practices. No warranty, either expressed or implied, is made. Our conclusions and recommendations are based on the preliminary design information furnished to us, the data obtained from the subsurface exploration program, and/or current geotechnical engineering practices. The findings and recommendations do not reflect variations in subsurface conditions that could exist between the boring locations or in unexplored areas of the site. Should such variations become apparent during construction, it will be necessary to re-evaluate our conclusions and recommendations based upon on-site observations of the conditions.

Regardless of thoroughness of a subsurface exploration, there is the possibility that conditions in other areas will differ from those at the boring locations and the conditions may not be as anticipated by the designers. Additionally, the construction process may alter the soil conditions. Therefore, experienced geotechnical engineers should evaluate earthwork and foundation construction to verify that the conditions anticipated in design actually exist in the field at the time of construction. Otherwise, we assume no responsibility for construction compliance with the design concepts, specifications, or recommendations.

In the event that changes are made in the design or location of the proposed facilities, the recommendations presented in the report shall not be considered valid unless the changes are reviewed by our firm and conclusions of this report modified and/or verified in writing.

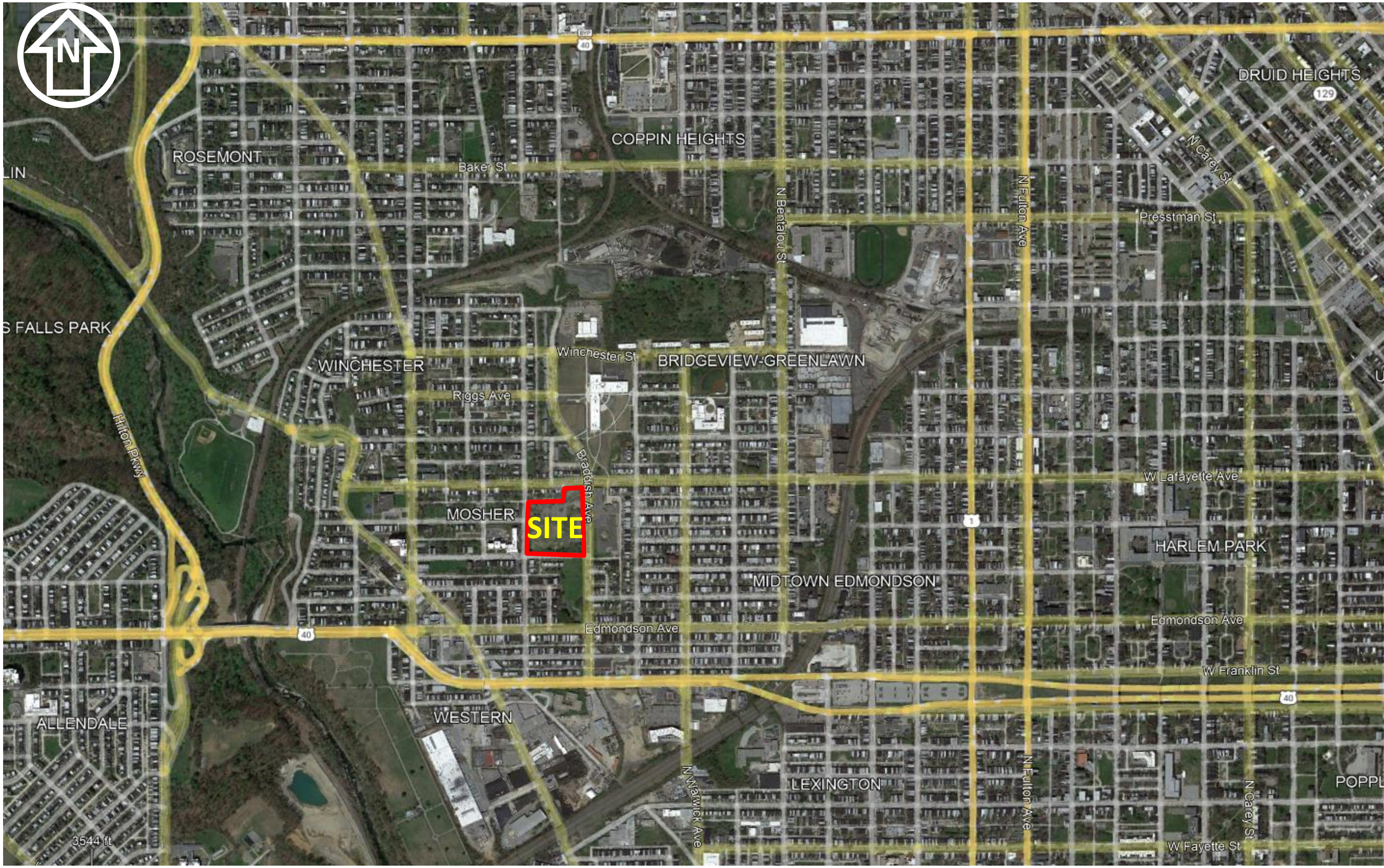
If this report is copied or transmitted to a third party, it must be copied or transmitted in its entirety, including text, attachments, and enclosures. Interpretations based on only a part of this report may not be valid.

It is important to note that our study was done in an effort to assist planning and design personnel in the preparation of generalized drawings and specifications for the project. As a result of this, potential contractors should be encouraged to conduct their own individually tailored studies to assess soils conditions, rock levels, excavation slope gradients, temporary excavation support methods, and groundwater/perched water levels and conditions. Specifically, our report has been prepared for generalized purposes of planning and design and may not be sufficiently comprehensive for bid preparation purposes.

APPENDIX A

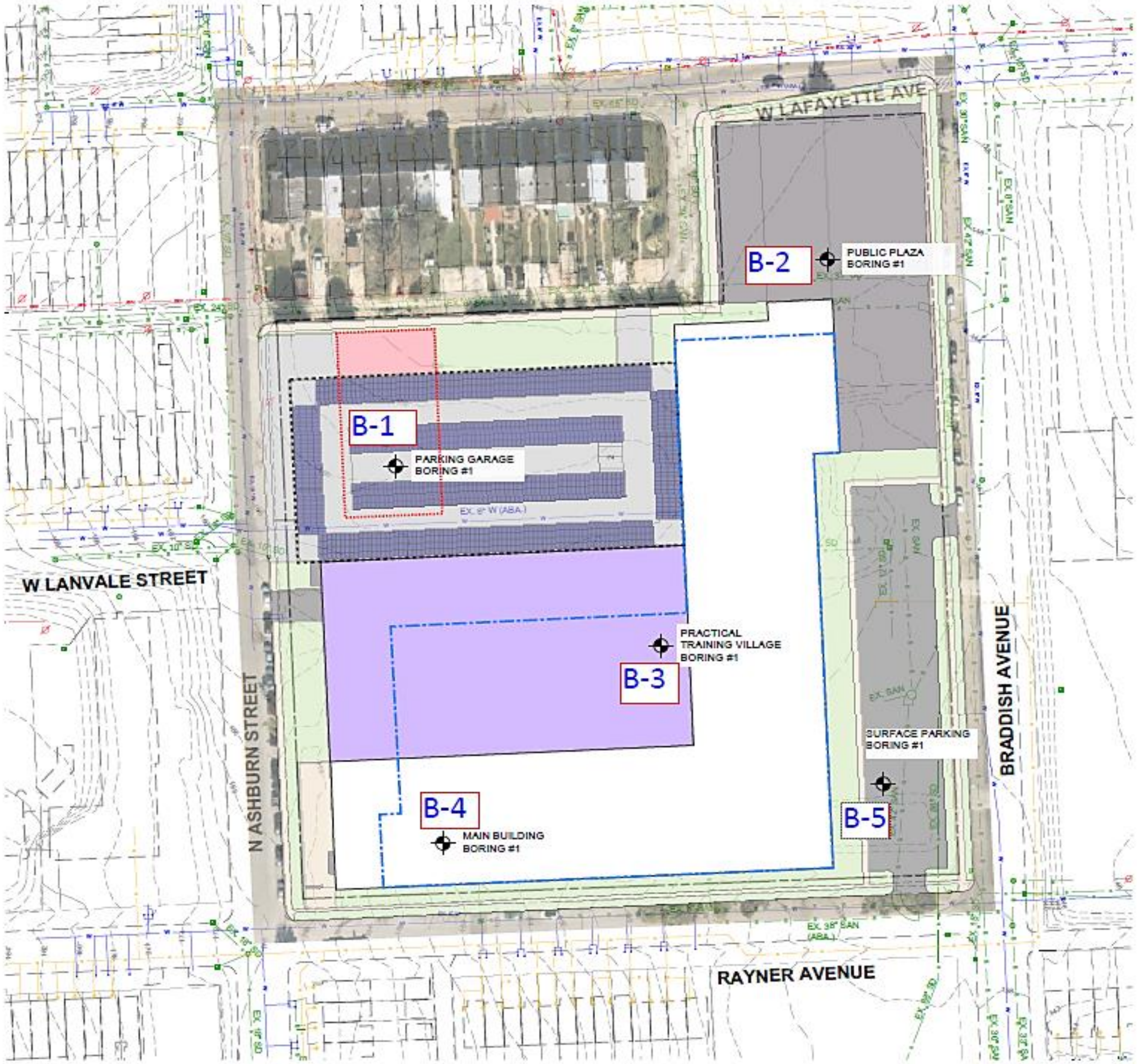
Site Location Plan

Boring Location Plan



Sourced by Google Map

KIM ENGINEERING 3916 VERO ROAD, SUITE K BALTIMORE, MD 21227	**SITE LOCATION PLAN** **COPPIN STATE UNIVERSITY PUBLIC SAFETY FACILITY** **SITE # 2 (RAYNER AVE AND BRADDISH AVE)** BALTIMORE, MD	PROJECT NO.: G22091 SCALE: NTS DATE: DECEMBER 8, 2022 **DRAWING NO. 1**



EX. ZONING: EC-2
EDUCATIONAL CAMPUS ZONING. A CAMPUS DISTRICT FOR COLLEGES AND UNIVERSITIES THAT ALLOWS FOR CERTAIN NON-EDUCATIONAL USES AND DORMITORIES FOR STUDENTS.

REQUIRED SETBACKS
NONE IF BUILDING DOES NOT ABUT ANOTHER ZONING DISTRICT OTHERWISE:
FRONT - 20'
INTERIOR SIDE - 10'
CORNER SIDE - 20'
REAR - 15'

REQUIRED PARKING FOR POST SECONDARY EDUCATIONAL FACILITY:
1 PER 4 EMPLOYEES ON PEAK SHIFT +
1 PER 25 STUDENTS BASED ON THE MAXIMUM STUDENT CAPACITY

LEGEND

MAIN BUILDING STRUCTURE
3 ABOVE GROUND STORIES

PRACTICAL TRAINING VILLAGE STRUCTURE
4 ABOVE GROUND STORIES

PARKING GARAGE STRUCTURE
5 ABOVE GROUND STORIES
2 BELOW GROUND STORIES

APPARATUS BAY STRUCTURE
1 ABOVE GROUND STORY

FIRING RANGE STRUCTURE
1 BELOW GROUND STORY

BORING LOCATIONS

KIM ENGINEERING
3916 VERO ROAD, SUITE K BALTIMORE, MD 21227

BORING LOCATION PLAN
COPPIN STATE UNIVERSITY PUBLIC SAFETY FACILITY
SITE # 2 (RAYNER AVE AND BRADDISH AVE)
BALTIMORE, MD

PROJECT NO.: G22091
SCALE: NTS
DATE: DECEMBER 8, 2022
DRAWING NO. 2

APPENDIX B

SUBSURFACE INVESTIGATION

Identification of Soil

Record of Soil Exploration Logs

KIM ENGINEERING

IDENTIFICATION OF SOIL

Soil Classification - ASTM D-2487

Coarse Grained Soils, More than 50% is retained on the No. 200 sieve	**Gravels -** More than 50% of the course fraction is retained on the No. 4 sieve. Coarse = 1" - 3" Medium = 1/2" - 1 " Fine = 1/4" to 1/2"	Clean Gravels <5% Passing No. 200 sieve	**GW**	Well Graded Gravel
			GP	Poorly Graded Gravel
		Gravels with fines >12% passing No. 200 sieve	**GM**	Silty Gravel
			GC	Clayey Gravel
	Sands - More than 50% of the coarse fraction passes the No.4 sieve Coarse = No. 10 to No. 4 Medium = No. 10 to No. 40 Fine = No. 40 to No. 200	Clean Sands <5% passing No. 200 sieve	**SW**	Well Graded Sand
			SP	Poorly Graded Sand
		Sands with fines >12% passing No. 200 sieve	**SM**	Silty Sand
			SC	Clayey Sand
Fine Grained Soils, More than 50% passes the No. 200 sieve	**Silts and Clays** Liquid Limit of 50 or less Low to medium plasticity	Inorganic	**ML**	Silt
			CL	Lean Clay
		Organic	**OL**	Organic silt
				Organin clay
	Silts and Clays Liquid limit of 50 or greater Medium to high plasticity	Inorganic	**MH**	Elastic silt
			CH	Fat clay
		Organic	**OH**	Organic silt
				Organic clay
Highly Organic	Primarily Organic matter, dark color, organic odor		**PT**	Peat

Terminology and Definitions

Portions of Soil Components		
Component Form	Description	Label
Noun	Gravel, Sand, Silt, Clay	50% or more
Adjective	Sandy, Silty, Clayey	35% to 49%
Some	some Sand, some Silt	12% to 34%
Trace	trace Sand, trace Clay	1% to 11%
With	with Sand, with Silt	presence only

Particle Size Identification	
Particle Size	Particle Dimension
Boulder	12" diamter or more
Cobble	3" to 12" diamter
Gravel	1/4" to 3" diamter
Sand	0.005" to 1/4" diamter
Silt/ Clay (fines)	Cannot See Particle

Cohesive Soils		
Field Description	N- Value	Consistency
Easily Molded in Hands	0-2	Very Soft
Easily Penetrated Several inches by thumb	3-4	Soft
Penetrated by thumb with Moderate Effort	5-8	Medium Stiff
Penetrated by Thumb with Great Effort	9-15	Stiff
Indented by Thumb with only Great Effort	16-30	very Stiff
Difficult to indent by thumbnail	> 30	Hard

Granular Soils	
N- Values	Relative Density
0-4	Very Loose
5-10	Loose
11-30	Medium Dense
31-50	Dense
Greater than 50	Very Dense

Fill: Man made deposit of soils, rock and waste material.
Probable Fill: Soils which contain no visually detected foreign matter but which may be man made deposit.
Rock Fragments: Angular Pieces of rock, distinguished from transported gravel, which have seperated from orginal wein or strata and are present in soil matrix.
Disintregrated Rock: Residual rock material with SPT of more than 60 blows per ft. and less than refusal.
Karst: Descriptive term which denotes the potential for solutioning of limestone rock and the development of sink holes.
Alluvium: Recently depositied soils placed by water action, typically stream or river flood plain soils.
Ironite: Iron oxide deposited within a soil layer forming cemented deposits.
Quarts: A hard silica mineral often found in residual soils.
Mica: A soft plate of silica mineral found in many rocks. And in residual or transported soil derived there from.
Layers: 1/2 to 12 inch seam of minor soil component.
Lenses: 0 to 1/2 inch seam of minor soil component.
Pocket: Discontinuous body of minor soil component.

Record of Soil Exploration Logs

KIM

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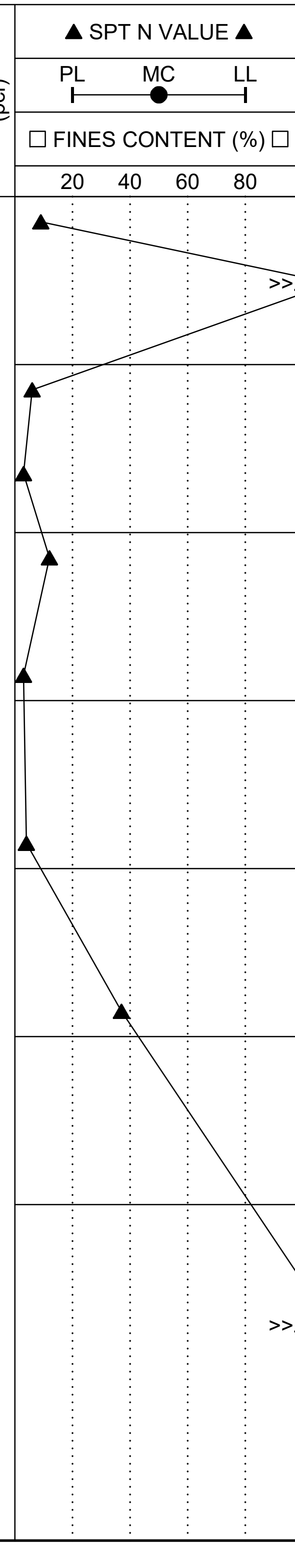
Baltimore, Maryland

BORING NUMBER B-1 (Site B)

PAGE 1 OF 1

CLIENT Manns Woodward Studios, Inc.
PROJECT NAME Coppin State University PSF
PROJECT NUMBER G22091
PROJECT LOCATION Baltimore, Maryland
DATE STARTED 11/29/22 **COMPLETED** 11/29/22
GROUND ELEVATION 155 ft **HOLE SIZE** 6"
DRILLING CONTRACTOR Kim Engineering Inc.
GROUND WATER LEVELS:
DRILLING METHOD H.S.A.
▽ **AT TIME OF DRILLING** 18.5 ft / Elev 136.5 ft
LOGGED BY SE **CHECKED BY** TL
▼ **AT END OF DRILLING** 14.3 ft / Elev 140.7 ft
NOTES Caved @ 22.0'
AFTER DRILLING ---

DEPTH (ft)	ELEVATION	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)
0	154.58	5-inches of Topsoil					
		Brown, grayish brown, gray, red, moist, silty Sand with concrete, coal and glass fragments. (FILL)	SS 1	33	1-4-5 (9)		
			SS 2	100	50/2"		
5			SS 3	78	4-3-3 (6)		
			SS 4	44	3-2-1 (3)		
10		Black, greenish gray, brown, sandy lean Clay.	SS 5	100	9-6-6 (12)		
15		Brown, white, silty Sand with mica.	SS 6	44	3-1-2 (3)		
20		Gray, dark gray, sandy Silt with glass fragments.	SS 7	67	2-2-2 (4)		
25	131.50	Brown, dark brown, moist, dense, silty SAND (SM) with mica.	SS 8	67	5-16-21 (37)		
30	126.50	DECOMPOSED ROCK classified as brown, orange, dark brown, black, moist, very dense, silty SAND (SM) with mica.	SS 9	100	50/6"		
	121.30	Bottom of hole at 33.7 feet.	SS 10	100	50/2"		



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Consulting Geotechinical Engineers
Baltimore, Maryland

BORING NUMBER B-2 (Site B)

CLIENT Manns Woodward Studios, Inc.
PROJECT NAME Coppin State University PSF
PROJECT NUMBER G22091
PROJECT LOCATION Baltimore, Maryland
DATE STARTED 11/29/22 **COMPLETED** 11/29/22
GROUND ELEVATION 147 ft **HOLE SIZE** 6"
DRILLING CONTRACTOR Kim Engineering Inc.
GROUND WATER LEVELS:
DRILLING METHOD H.S.A.
∇ **AT TIME OF DRILLING** 23.5 ft / Elev 123.5 ft
LOGGED BY SE **CHECKED BY** TL
AT END OF DRILLING ---
NOTES Caved @ 3'
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	ELEVATION	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ / PL MC LL / ☐ FINES CONTENT (%) ☐ (20 40 60 80)
0		146.67	4-inches of Topsoil.	SS 1	56	4-4-4 (8)			
			Brown, dark brown, white, black, moist, silty Sand with asphalt and brick fragments. (FILL)	SS 2	78	4-5-6 (11)			
5				SS 3	78	3-4-6 (10)			
				SS 4	67	1-1-1 (2)			
10				SS 5	56	1-2-2 (4)			
15				SS 6	56	2-2-2 (4)			
20				SS 7	44	1-1-2 (3)			
25		∇	Gray, dark gray, black, sandy Silt with glass fragments.	SS 8	67	2-2-2 (4)			
30		118.50	Gray, dark gray, black,moist, medium stiff, sandy SILT (ML).	SS 9	72	2-3-3 (6)			
		113.50	Decomposed rock classified as grayish brown, wet, very dense, silty SAND (SM).	SS 10	100	50/1"			>>
		113.42	Bottom of hole at 33.6 feet.						

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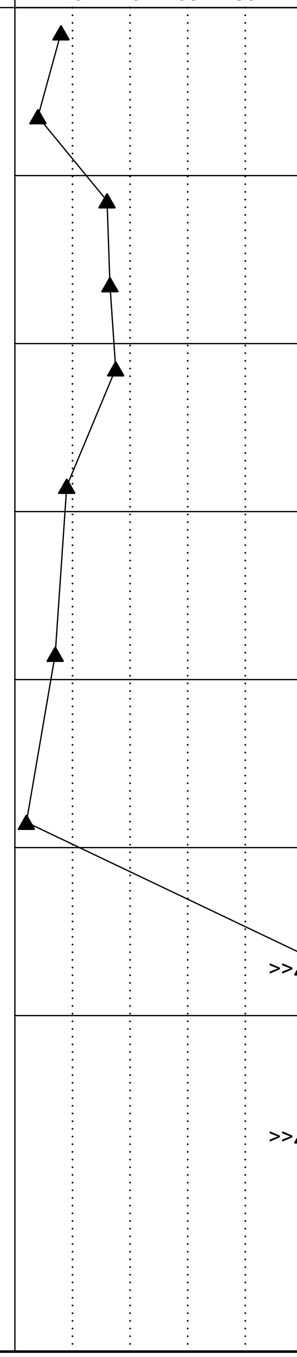
Baltimore, Maryland

BORING NUMBER B-3 (Site B)

CLIENT Manns Woodward Studios, Inc.
PROJECT NAME Coppin State University PSF
PROJECT NUMBER G22091
PROJECT LOCATION Baltimore, Maryland
DATE STARTED 11/28/22 **COMPLETED** 11/28/22
GROUND ELEVATION 144 ft **HOLE SIZE** 6"
DRILLING CONTRACTOR Kim Engineering Inc.
GROUND WATER LEVELS:
DRILLING METHOD H.S.A.
AT TIME OF DRILLING ---
LOGGED BY SE **CHECKED BY** TL
AT END OF DRILLING 23.5 ft / Elev 120.5 ft
NOTES
24hrs AFTER DRILLING 19.0 ft / Elev 125.0 ft

DEPTH (ft)	ELEVATION	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)
0		6-inches of Topsoil.	SS 1	44	3-8-8 (16)		
	143.50	Brown, grayish brown, white, black, moist, silty Sand with brick, asphalt and concrete fragments. (FILL)	SS 2	56	6-4-4 (8)		
5			SS 3	67	8-13-19 (32)		
			SS 4	56	35-23-10 (33)		
10			SS 5	56	21-20-15 (35)		
15			SS 6	72	13-8-10 (18)		
20		silty Sand with ash and brick fragments.	SS 7	56	4-8-6 (14)		
25	120.50	Brown, light brown, gray, moist, soft, sandy SILT (ML) with mica.	SS 8	72	2-2-2 (4)		
30	115.50	DECOMPOSED ROCKS classified as brown, moist, very dense, silty SAND (SM) with mica and decomposed rock fragments.	SS 9	100	50/2"		
	110.30	Bottom of hole at 33.7 feet.	SS 10	100	50/2"		

▲ SPT N VALUE ▲
PL MC LL
☐ FINES CONTENT (%) ☐
20 40 60 80



KIM

KIM ENGINEERING, INC.
Consulting Geotechinical Engineers
Baltimore, Maryland

BORING NUMBER B-4 (Site B)

CLIENT Manns Woodward Studios, Inc.
PROJECT NAME Coppin State University PSF
PROJECT NUMBER G22091
PROJECT LOCATION Baltimore, Maryland
DATE STARTED 11/29/22 **COMPLETED** 11/29/22
GROUND ELEVATION 156 ft **HOLE SIZE** 6"
DRILLING CONTRACTOR Kim Engineering Inc.
GROUND WATER LEVELS:
DRILLING METHOD H.S.A.
AT TIME OF DRILLING 33.5 ft / Elev 122.5 ft
LOGGED BY SE **CHECKED BY** TL
AT END OF DRILLING 29.0 ft / Elev 127.0 ft
NOTES
24hrs AFTER DRILLING 28.0 ft / Elev 128.0 ft

DEPTH (ft)	ELEVATION	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	SPT N VALUE
0	155.62	4.5-inches of Topsoil.	SS 1	33	1-2-24 (26)			
		Brown, grayish brown, light brown, dark gray, black, moist, silty Sand with concrete and brick fragments. (FILL)	SS 2	56	7-7-8 (15)			
5			SS 3	25	16-44-50/4"			>>
			SS 4	44	10-22-40 (62)			
10	146.00	DECOMPOSED ROCK classified as brown, light brown, gray, white, moist to wet, very dense, silty SAND (SM) with mica and gravel.	SS 5	75	50/4"			>>
15			SS 6	75	50/4"			>>
20			SS 7	100	50/2"			>>
25			SS 8	100	50/2"			>>
30			SS 9	100	50/2"			>>
	122.42	Bottom of hole at 33.6 feet.	SS 10	100	50/1"			>>

SPT N VALUE; PL, MC, LL; FINES CONTENT (%); 20 40 60 80

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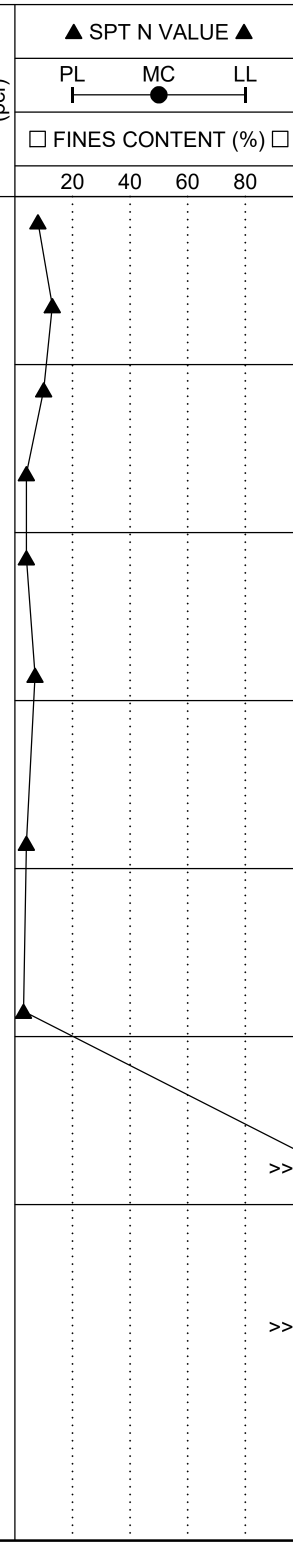
Consulting Geotechinical Engineers
Baltimore, Maryland

BORING NUMBER B-5 (Site B)

CLIENT Manns Woodward Studios, Inc.
PROJECT NAME Coppin State University PSF
PROJECT NUMBER G22091
PROJECT LOCATION Baltimore, Maryland
DATE STARTED 12/1/22 **COMPLETED** 12/1/22
GROUND ELEVATION 140 ft **HOLE SIZE** 6"
DRILLING CONTRACTOR Kim Engineering Inc.
GROUND WATER LEVELS:
DRILLING METHOD H.S.A.
AT TIME OF DRILLING 18.5 ft / Elev 121.5 ft
LOGGED BY SE **CHECKED BY** TL
AT END OF DRILLING ---
NOTES Caved @ 29.16'
AFTER DRILLING ---

DEPTH (ft)	ELEVATION	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)
0	139.58	5-inches of topsoil.					
		Black, white, dark gray, dark brown, moist to wet, silty Sand with concrete and brick fragments. (FILL)	SS 1	44	4-4-4 (8)		
			SS 2	56	9-7-6 (13)		
5			SS 3	56	8-4-6 (10)		
			SS 4	56	2-2-2 (4)		
10			SS 5	44	2-2-2 (4)		
15			SS 6	44	4-5-2 (7)		
20			SS 7	67	2-2-2 (4)		
	116.50	Grayish brown, greenish gray, very loose, moist to wet, silty SAND (SM) with trace of gravel.	SS 8	56	2-1-2 (3)		
25							
	111.50	DECOMPOSED ROCK classified as brown, orangish brown, gray, moist, very dense, silty SAND (SM) with gravel.	SS 9	40	9-50/4"		
30							
	106.25	Bottom of hole at 33.8 feet.	SS 10	67	50/3"		



APPENDIX C

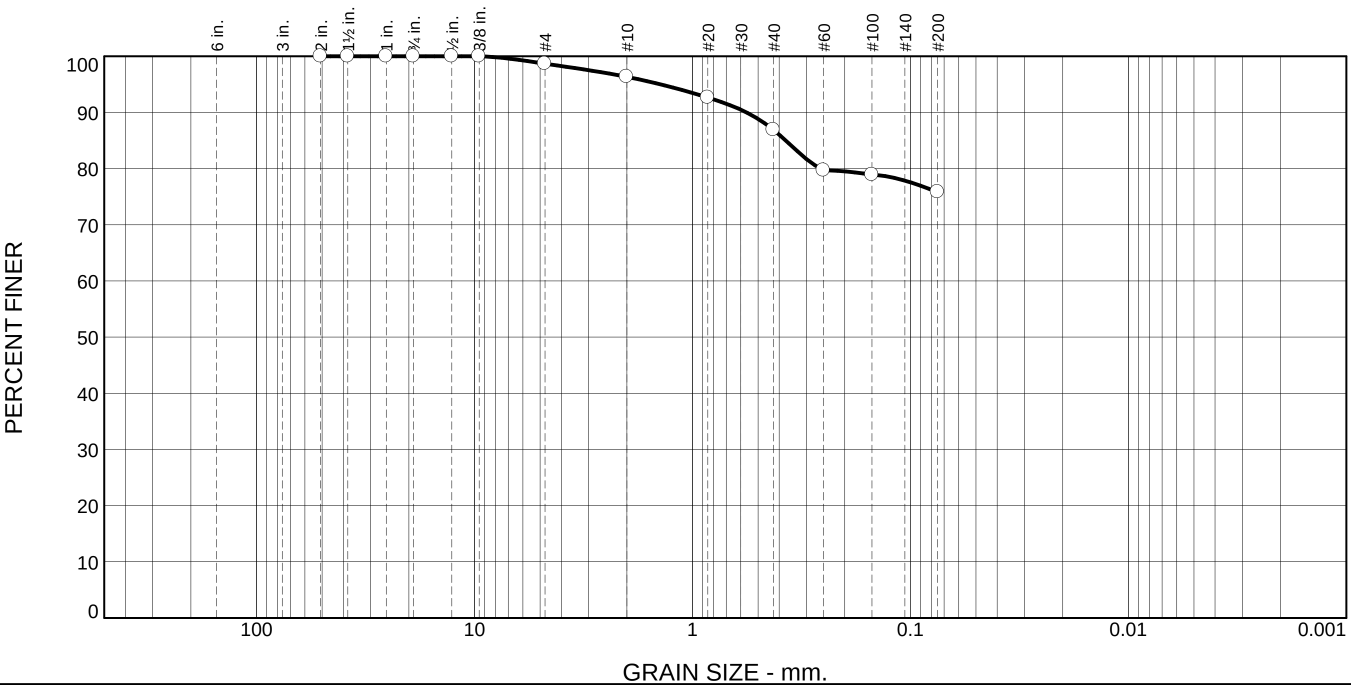
GEOTECHNICAL LABORATORY TESTS

Particle Size Distribution Report

Liquid Limit and Plastic Limit Report

Particle Size Distribution Report

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	1.3	2.4	9.4	11.1	75.8	

TEST RESULTS			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
2"	100.0		
1 1/2"	100.0		
1"	100.0		
3/4"	100.0		
1/2"	100.0		
3/8"	100.0		
#4	98.7		
#10	96.3		
#20	92.6		
#40	86.9		
#60	79.7		
#100	78.9		
#200	75.8		

* (no specification provided)

Material Description

lean clay with sand (CL)

Atterberg Limits (ASTM D 4318)

PL= 17 **LL=** 30 **PI=** 13

Classification

USCS (D 2487)= CL **AASHTO (M 145)=** A-6(8)

Coefficients

D_{90}= 0.5650 D_{85}= 0.3732 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 11/29/2022 **Date Tested:** 12/09/2022

Tested By: KS

Checked By: TL

Title: Principal Engineer

Source of Sample: B-1 **Depth:** 10'00"-11'6" **Date Sampled:** 11/29/2022
Sample Number: S-5

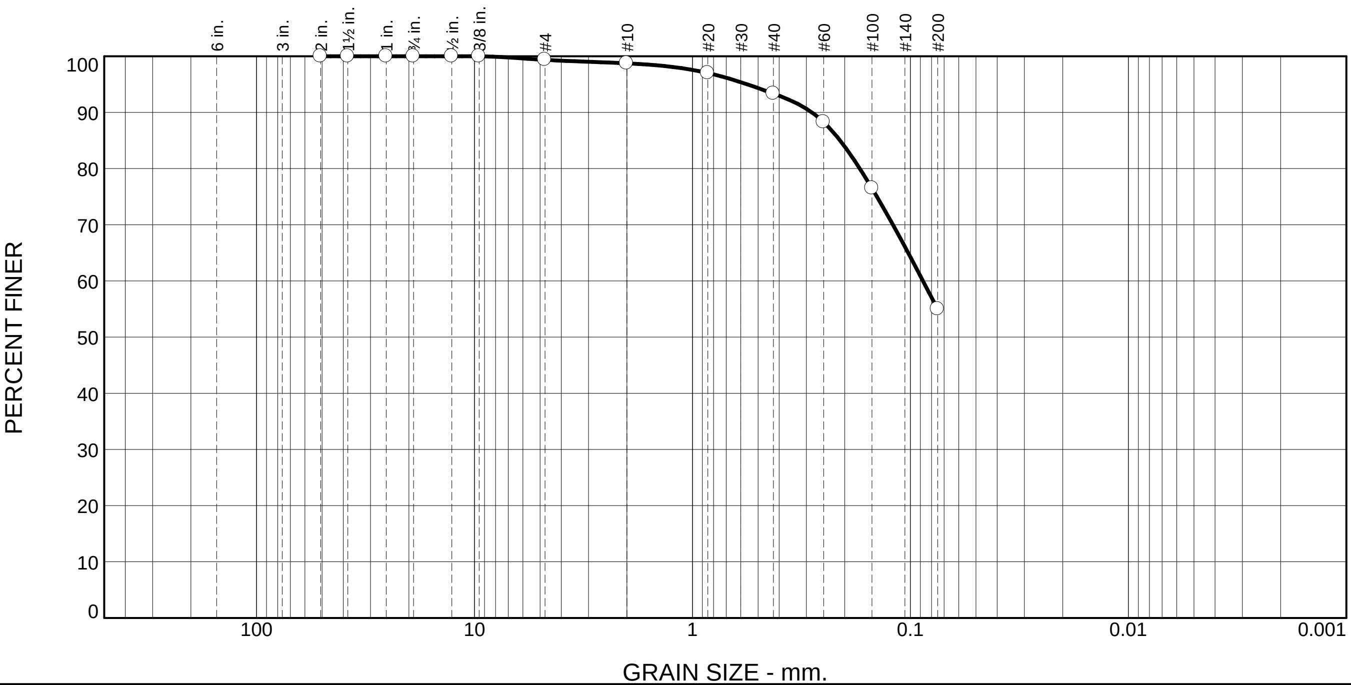
KIM ENGINEERING, INC.

Beltsville, Maryland

Client: Manns Woodward Studios, Inc.
Project: Coppin State University PSF Site B

Project No: G22091 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.6	0.7	5.4	38.3	55.0	

TEST RESULTS

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
2"	100.0		
1 1/2"	100.0		
1"	100.0		
3/4"	100.0		
1/2"	100.0		
3/8"	100.0		
#4	99.4		
#10	98.7		
#20	97.0		
#40	93.3		
#60	88.3		
#100	76.5		
#200	55.0		

* (no specification provided)

Material Description

sandy silt (ML)

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= ML **AASHTO (M 145)=** A-4(0)

Coefficients

D_{90}= 0.2832 D_{85}= 0.2101 D_{60}= 0.0876
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 11/29/2022 **Date Tested:** 12/09/2022
Tested By: KS
Checked By: TL
Title: Principal Engineer

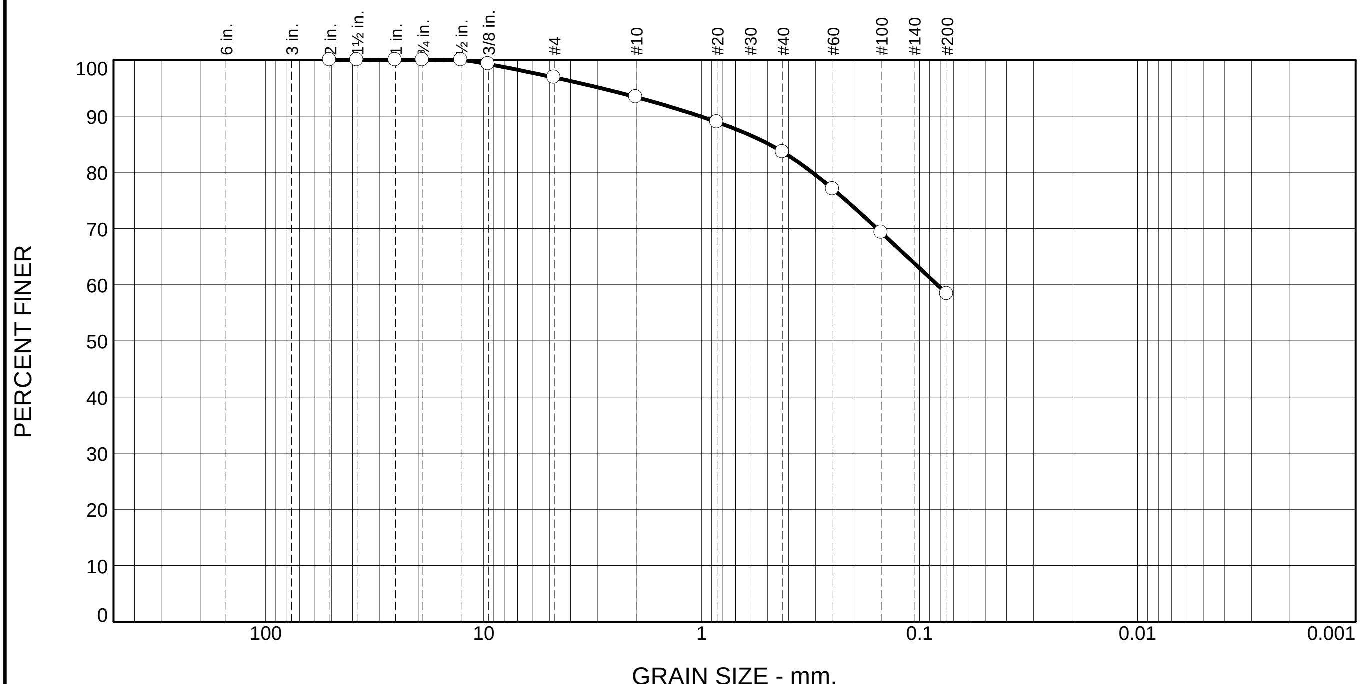
Source of Sample: B-2 **Depth:** 28'6"-30'00" **Date Sampled:** 11/29/2022
Sample Number: S-9

KIM ENGINEERING, INC.

Beltsville, Maryland

Client: Manns Woodward Studios, Inc.
Project: Coppin State University PSF Site B
Project No: G22091 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	3.1	3.5	9.8	25.2	58.4	

TEST RESULTS

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
2"	100.0		
1 1/2"	100.0		
1"	100.0		
3/4"	100.0		
1/2"	100.0		
3/8"	99.3		
#4	96.9		
#10	93.4		
#20	88.9		
#40	83.6		
#60	77.0		
#100	69.3		
#200	58.4		

* (no specification provided)

Material Description

sandy silt (ML)

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= ML **AASHTO (M 145)=** A-4(0)

Coefficients

D_{90}= 1.0204 D_{85}= 0.4911 D_{60}= 0.0831
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 11/28/2022 **Date Tested:** 12/09/2022
Tested By: KS
Checked By: TL
Title: Principal Engineer

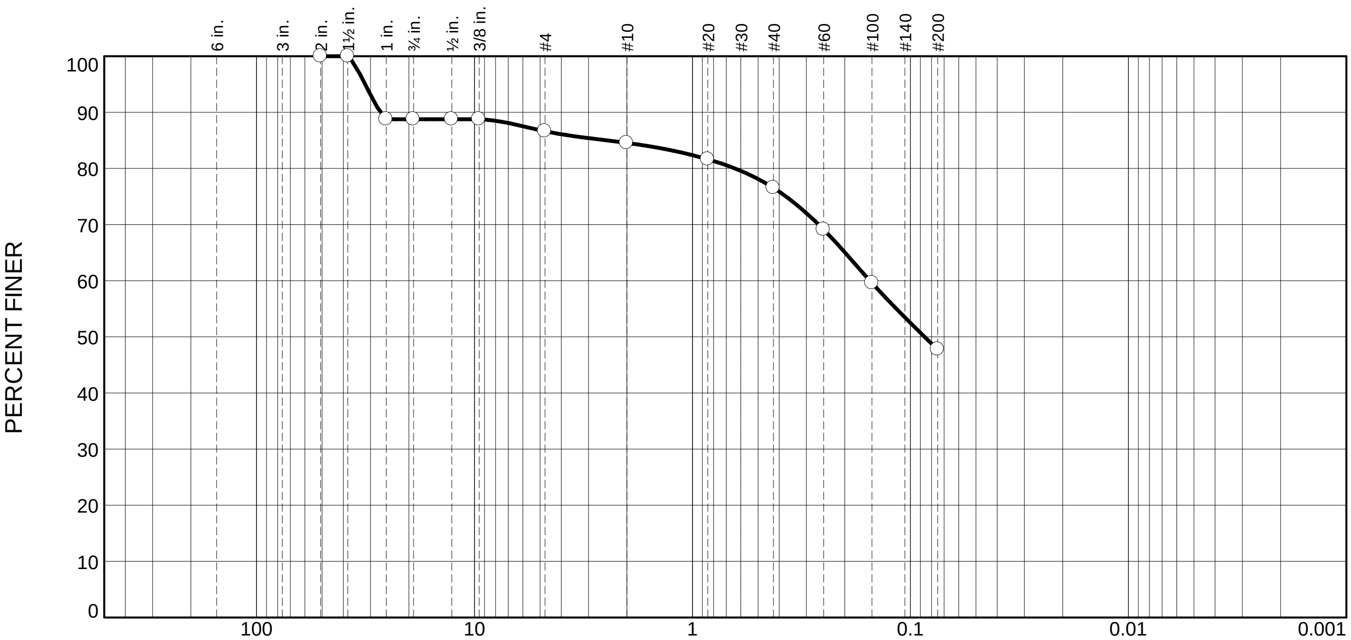
Source of Sample: B-3 **Depth:** 23'6"-25'00" **Date Sampled:** 11/28/2022
Sample Number: S-8

KIM ENGINEERING, INC.

Beltsville, Maryland

Client: Manns Woodward Studios, Inc.
Project: Coppin State University PSF Site B
Project No: G22091 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	11.2	2.2	2.1	8.0	28.7	47.8	

TEST RESULTS

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
2"	100.0		
1 1/2"	100.0		
1"	88.8		
3/4"	88.8		
1/2"	88.8		
3/8"	88.8		
#4	86.6		
#10	84.5		
#20	81.6		
#40	76.5		
#60	69.1		
#100	59.6		
#200	47.8		

* (no specification provided)

Material Description

silty sand (SM)

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= SM **AASHTO (M 145)=** A-4(0)

Coefficients

D_{90}= 27.0038 D_{85}= 2.4741 D_{60}= 0.1533
D_{50}= 0.0860 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 12/01/2022 **Date Tested:** 12/09/2022

Tested By: KS

Checked By: TL

Title: Principal Engineer

Source of Sample: B-5 **Depth:** 23'6"-25'00" **Date Sampled:** 12/01/2022
Sample Number: S-8

KIM ENGINEERING, INC.

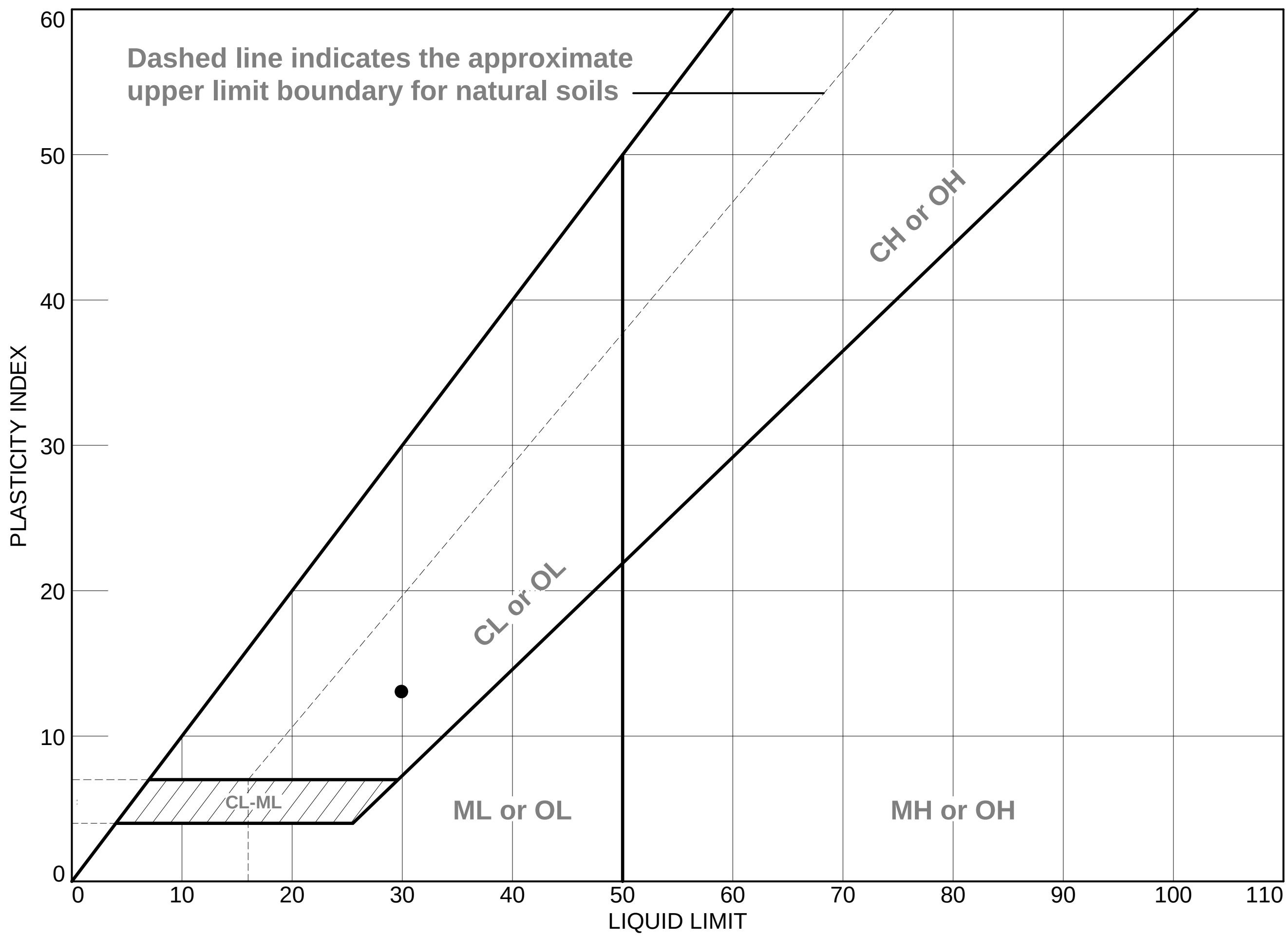
Beltsville, Maryland

Client: Manns Woodward Studios, Inc.
Project: Coppin State University PSF Site B

Project No: G22091 **Figure**

Liquid Limit and Plastic Limit Report

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	B-1	S-5	10'00"-11'6"	17.6	17	30	13	CL

KIM ENGINEERING, INC.

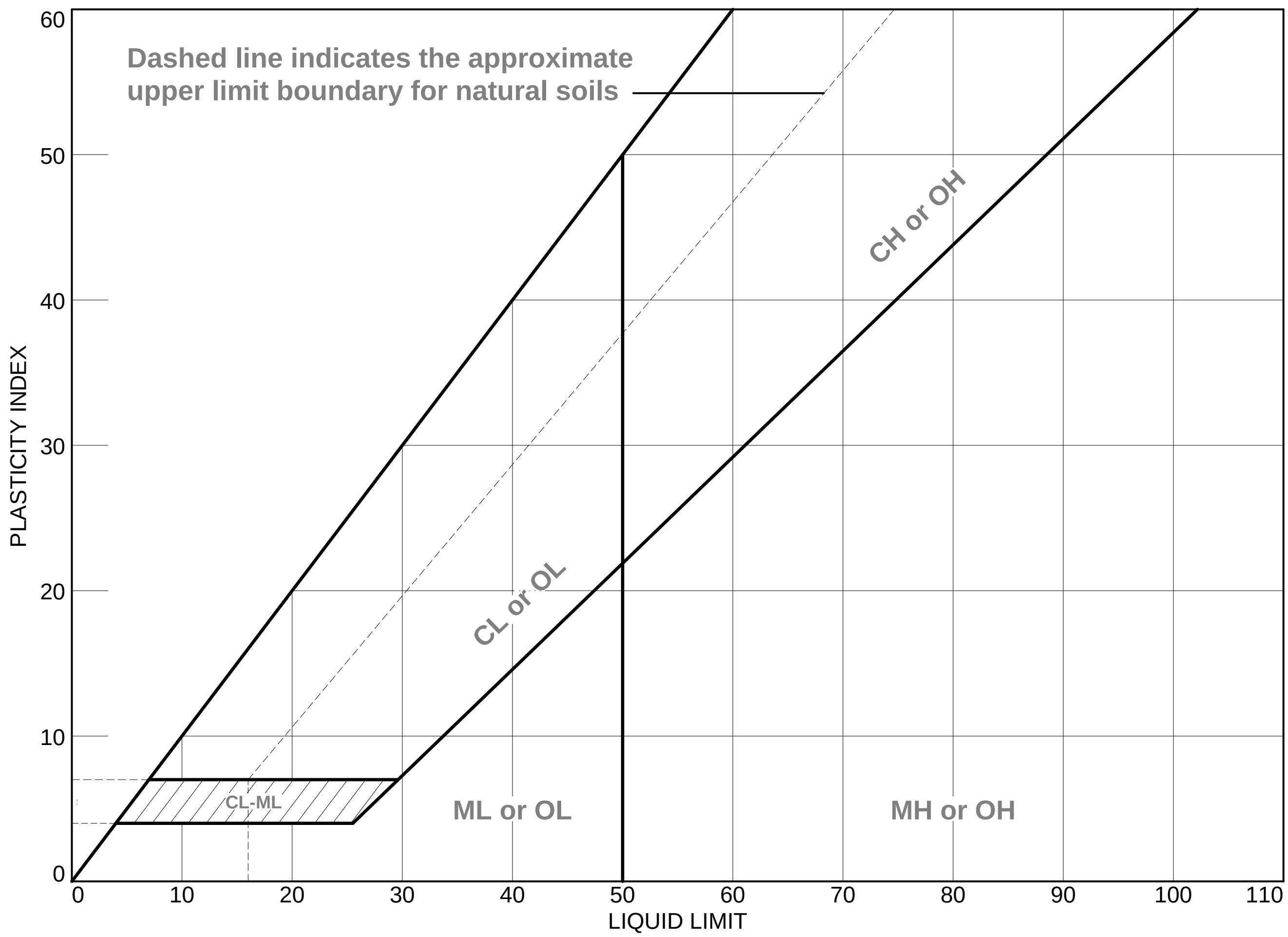
Beltsville, Maryland

Client: Manns Woodward Studios, Inc.
Project: Coppin State University PSF Site B

Project No.: G22091 **Figure**

Tested By: KS **Checked By:** TL

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	**SOURCE**	**SAMPLE NO.**	**DEPTH**	**NATURAL WATER CONTENT (%)**	**PLASTIC LIMIT (%)**	**LIQUID LIMIT (%)**	**PLASTICITY INDEX (%)**	**USCS**
●	B-2	S-9	28'6"-30'00"	33.8	NP	NV	NP	ML

KIM ENGINEERING, INC.

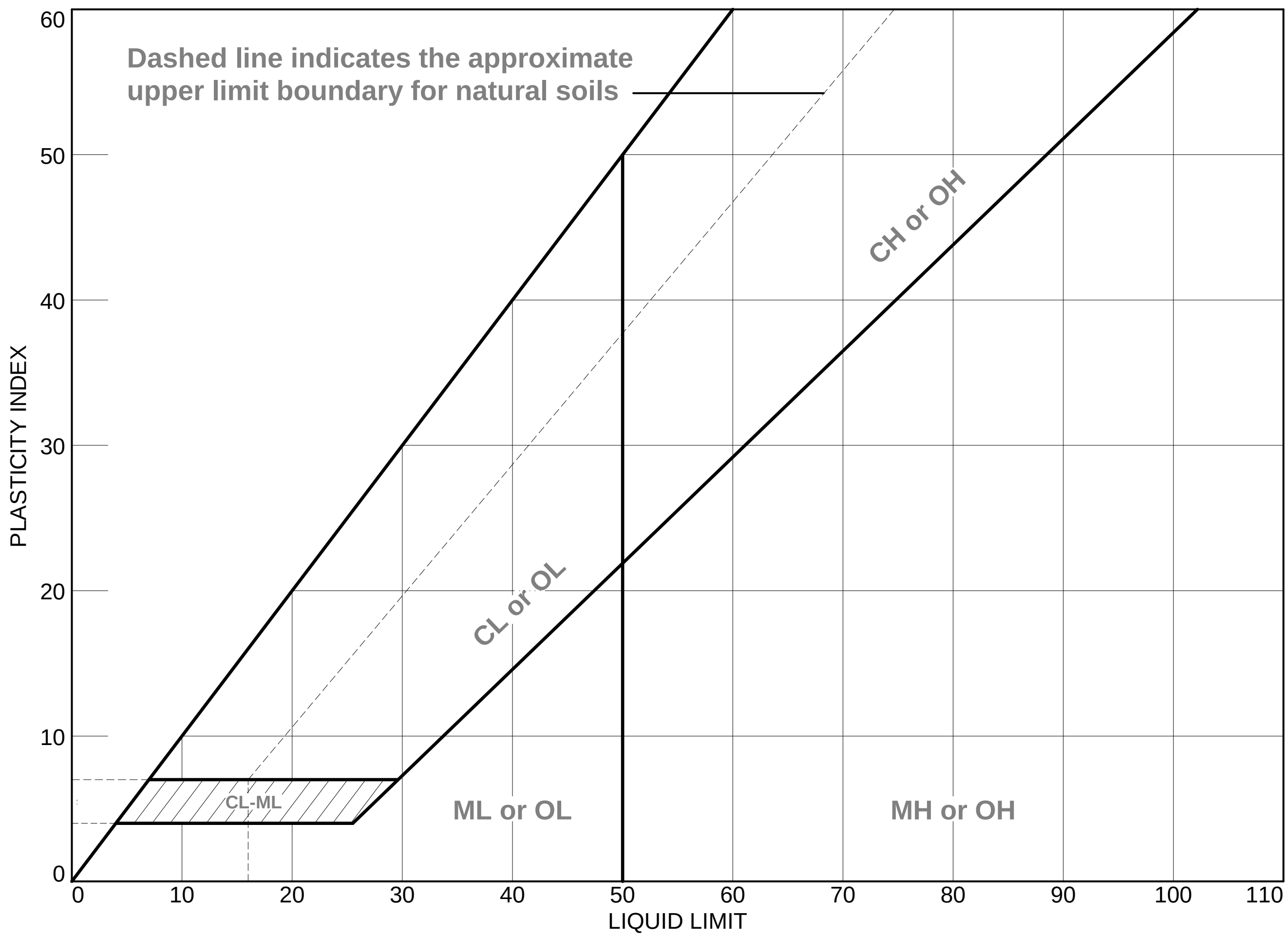
Beltsville, Maryland

Client: Manns Woodward Studios, Inc.
Project: Coppin State University PSF Site B

Project No.: G22091 **Figure**

Tested By: KS **Checked By:** TL

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	B-3	S-8	23'6"-25'00"	39.2	NP	NV	NP	ML

KIM ENGINEERING, INC.

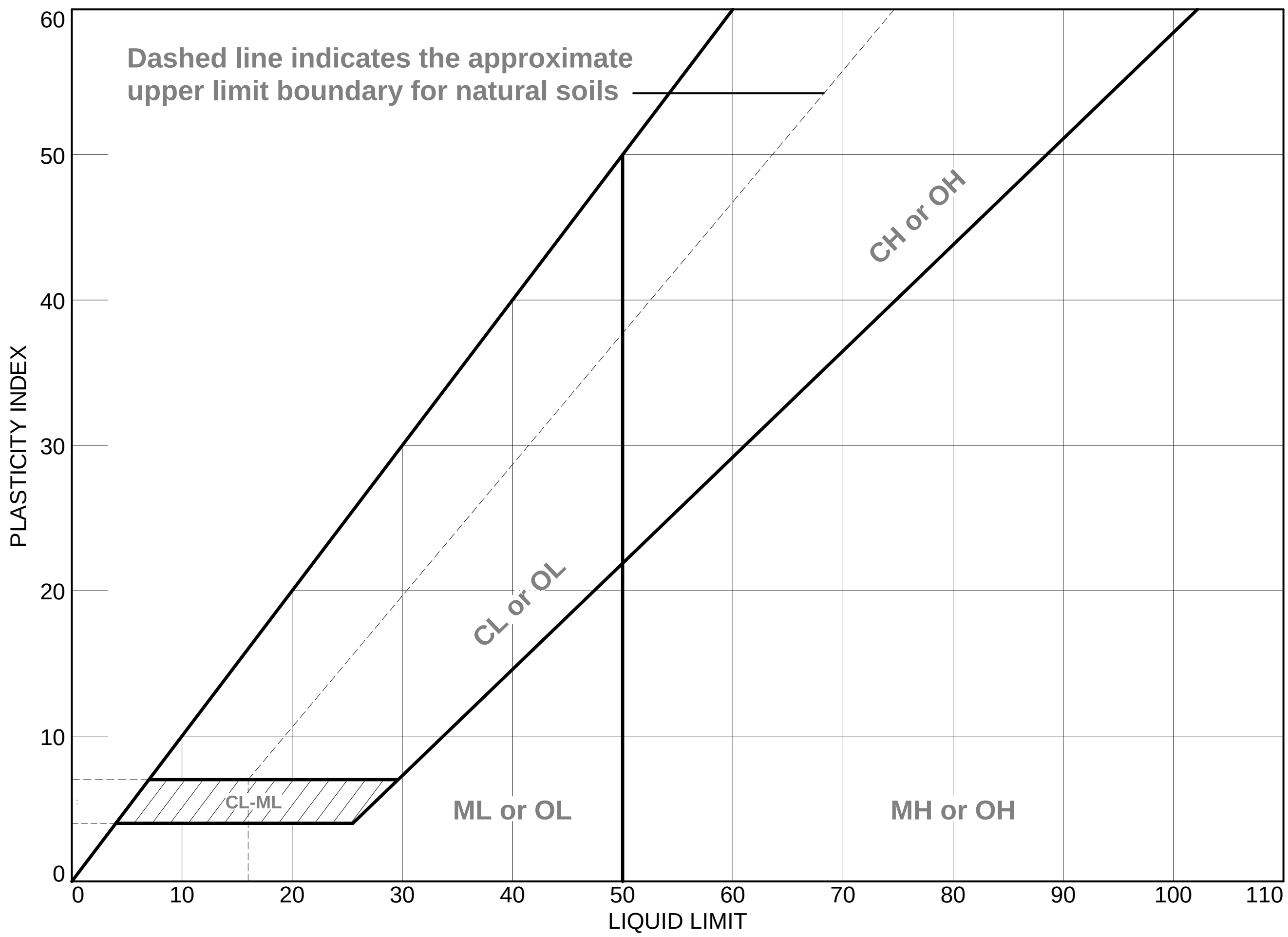
Beltsville, Maryland

Client: Manns Woodward Studios, Inc.
Project: Coppin State University PSF Site B

Project No.: G22091 **Figure**

Tested By: KS **Checked By:** TL

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	B-5	S-8	23'6"-25'00"	23.4	NP	NV	NP	SM

KIM ENGINEERING, INC.

Beltsville, Maryland

Client: Manns Woodward Studios, Inc.
Project: Coppin State University PSF Site B

Project No.: G22091 **Figure**

Tested By: KS **Checked By:** TL

APPENDIX D

SEISMIC DESIGN PARAMETERS

Coppin State University (Site 2)

Latitude, Longitude: 39.29693928, -76.66048985

Date	12/12/2022, 3:47:02 PM
Design Code Reference Document	ASCE7-16
Risk Category	III
Site Class	C - Very Dense Soil and Soft Rock

Type	Value	Description
S_S	0.14	MCE_R ground motion. (for 0.2 second period)
S_1	0.043	MCE_R ground motion. (for 1.0s period)
S_{MS}	0.183	Site-modified spectral acceleration value
S_{M1}	0.065	Site-modified spectral acceleration value
S_{DS}	0.122	Numeric seismic design value at 0.2 second SA
S_{D1}	0.043	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	A	Seismic design category
F_a	1.3	Site amplification factor at 0.2 second
F_v	1.5	Site amplification factor at 1.0 second
PGA	0.074	MCE_G peak ground acceleration
F_{PGA}	1.3	Site amplification factor at PGA
PGA_M	0.096	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	0.14	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	0.149	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.043	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.046	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.5	Factored deterministic acceleration value. (Peak Ground Acceleration)
PGA_{UH}	0.074	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.943	Mapped value of the risk coefficient at short periods
C_{R1}	0.928	Mapped value of the risk coefficient at a period of 1 s
C_V	0.7	Vertical coefficient

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Coppin State University (Site 2)

Latitude, Longitude: 39.29693928, -76.66048985

Date	12/12/2022, 3:48:47 PM
Design Code Reference Document	ASCE7-16
Risk Category	III
Site Class	D - Stiff Soil

Type	Value	Description
S_S	0.14	MCE_R ground motion. (for 0.2 second period)
S_1	0.043	MCE_R ground motion. (for 1.0s period)
S_{MS}	0.225	Site-modified spectral acceleration value
S_{M1}	0.103	Site-modified spectral acceleration value
S_{DS}	0.15	Numeric seismic design value at 0.2 second SA
S_{D1}	0.069	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	B	Seismic design category
F_a	1.6	Site amplification factor at 0.2 second
F_v	2.4	Site amplification factor at 1.0 second
PGA	0.074	MCE_G peak ground acceleration
F_{PGA}	1.6	Site amplification factor at PGA
PGA_M	0.118	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	0.14	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	0.149	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.043	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.046	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.5	Factored deterministic acceleration value. (Peak Ground Acceleration)
PGA_{UH}	0.074	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.943	Mapped value of the risk coefficient at short periods
C_{R1}	0.928	Mapped value of the risk coefficient at a period of 1 s
C_V	0.7	Vertical coefficient

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