PRELIMINARY GEOTECHNICAL INVESTIGATION REPORT
DRP RACER GENESEE INDUSTRIAL LAND

GENESEE TOWNSHIP, MICHIGAN

MSG PROJECT NO.: 401.2401267.000

JANUARY 2025

PREPARED FOR:

DETROIT REGIONAL PARTNERSHIP
1001 WOODWARD AVENUE, SUITE 800
DETROIT, MICHIGAN 48226

PREPARED BY:

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Mannik Smith GROUP

January 27, 2025

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Detroit Regional Partnership
1001 Woodward Ave Suite 800
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Detroit, Michigan 48226

RE: **Preliminary Geotechnical Investigation Report**
DRP RACER Genesee Industrial Land
Genesee Township, Michigan
MSG Project Number: 401.2401267.000

Dear Ms. Selby and Mr. Black,

This report presents the results of our preliminary geotechnical investigation for the site located on East Stanley Road in Genesee Township, Michigan. We completed this investigation in general accordance with MSG Proposal No. 401.2402074.OP0 dated October 10, 2024 and our agreement with Detroit Regional Partnership for professional services. This report presents our preliminary geotechnical recommendations and construction considerations for the project.

We trust that this report addresses your current project needs. We appreciate the opportunity to work with you on this very important project. Please contact us if you have any questions or if we can be of further assistance.

Sincerely,
The Mannik & Smith Group, Inc.

Kevin D Brown

John Ivoke, Ph.D.
Geotechnical Engineer

Kevin D. Brown, PE
Geotechnical Engineer

cc: Ms. Shannon Selby, Detroit Regional Partnership
Mr. Steven Black, RACER Trust

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EXECUTIVE SUMMARY

The Mannik & Smith Group, Inc., (MSG) was retained by the Detroit Regional Partnership (DRP) through the Verified Industrial Properties (VIP) program to conduct a preliminary geotechnical investigation supporting the physical site studies at the property owned by RACER Trust located on East Stanley Road in Genesee Township, Michigan.

The subsurface investigation consisted of performing a total of four (4) soil borings designated as SB-01 to SB-04; each boring was advanced to a depth of 25 feet below existing grade. All borings were advanced using a track-mounted Geoprobe 3230DT drill rig and advanced by hydraulically pushing 3.25-inch inner diameter steel casing.

Approximately 8 to 9 inches of topsoil was encountered at all the soil boring locations. Stratum 1 was encountered below the topsoil and consisted of stiff to hard lean clay extending to depths of 25 feet below grade.

Based upon our review of the existing soil conditions at the site, the subgrade soils consist of very stiff to hard lean clay at the minimum foundation bearing depth of 3.5 feet below existing grades. Provided the subgrade of the footings is properly prepared during construction, it is recommended that conventional shallow foundations bearing on the encountered native very stiff to hard clay material be designed for a net allowable bearing capacity of 4,000 psf. By utilizing proper construction techniques, the total settlement underneath the proposed shallow foundations in the native clay soil is anticipated to be less than 1 inch and the differential settlement is expected to be ¾ of the total settlement.

Based upon our review of the existing soil conditions at the site, an estimated modulus for subgrade reaction on top of the existing subgrade of 135 pounds per cubic inch (pci) may be used for slab-on-grade design. For a subgrade composed of well-compacted engineered fill a modulus of subgrade reaction of 175 pci may be used.

Based on the soil characteristics from the preliminary geotechnical investigation, a design CBR value of 5 and resilient modulus of 6,000 pci was estimated. Recommendations for both flexible and rigid pavement sections are presented in Section 4.3.

According to ASCE 7-22 Table 20.2-1, the proposed Site is designated as “Site Class CD” based on the average soil shear strength for the upper 25 feet of soil.

For standard shallow excavations, significant problems associated with groundwater seepage into the excavation are not anticipated; however, the Contractor should be prepared to address localized groundwater accumulation, perched groundwater, or surface run off (i.e., pumping water from prepared sumps).

This summary briefly discusses major findings covered within the body of the report. The intent of this executive summary is to provide a general summary. The report must be read carefully in its entirety before using any recommendations described herein.

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APPENDICES

1.0 INTRODUCTION

1.1 General

The Mannik & Smith Group, Inc., (MSG) was retained by the Detroit Regional Partnership (DRP) through the Verified Industrial Properties (VIP) program to conduct a preliminary geotechnical investigation supporting the physical site studies at the property owned by RACER Trust located on East Stanley Road in Genesee Township, Michigan. The approximate site location of the geotechnical study is depicted in Appendix A. This geotechnical investigation was performed in general accordance with MSG Proposal No. 401.2402074.OP0 dated October 10, 2024.

This preliminary geotechnical investigation report addresses feasible options for foundation design and pavement support. Once additional design details become available, including the depth and configurations of proposed structures, a supplemental geotechnical investigation should be completed to confirm encountered conditions and verify the preliminary recommendations contained herein.

1.2 Project Information and Site Conditions

As we understand, the scope of the preliminary geotechnical investigation is to provide a basic understanding of the geologic conditions within the developable areas of the site. The site is located on the north side of East Stanley Road east of the existing railroad, and approximately 0.4 miles west North Dort Highway. The site includes an area of approximately 80 acres, with Consumer's Energy property crossing through the site approximately 1,300 feet north of East Stanley Road, dividing the southern 15 acres with the rest of the site, which can be accessed through an existing easement. Costello drain runs east to west across the northern portion of the site.

The existing site topography elevation varied across the site ranging from approximately 765 feet to 810 feet. At the time of the preliminary investigation, detailed site grading plans and structural loading information for the proposed development were not available; however, we assume no significant grading (greater than 3 feet cut/fill) is anticipated at the site associated with this project.

2.0 SUBSURFACE INVESTIGATION

2.1 Field Exploration

The subsurface investigation consisted of performing a total of four (4) soil borings designated as SB-01 to SB-04; each boring was advanced to a depth of 25 feet below existing grade. The boring locations were field marked by MSG personnel, and the soil boring as-drilled coordinates were collected using a hand-held GPS unit. Elevations at the boring locations were estimated using Google Earth ™. A Soil Boring Location Plan is presented in Figure 2 in Appendix A.

The drilling operations were performed on December 12, 2024. All borings were advanced using a track-mounted Geoprobe 3230DT drill rig and advanced by hydraulically pushing 3.25-inch inner diameter steel casing. Upon completion, the boreholes were backfilled using soil cuttings.

Standard Penetration Tests (SPT) were conducted in accordance with ASTM D1586 ("Standard Method for Penetration Tests and Split Barrel Sampling of Soils") procedures and were generally completed at 2.5 feet intervals for the first 10 feet and at 5 feet intervals thereafter for all soil borings. Soil samples were recovered using the split-spoon sampling procedure. All soil samples were sealed in glass jars in the field to protect the soil and maintain the soil's natural moisture content, and the jars were labeled with the soil boring designation and a unique sample number. All samples were transferred to MSG's laboratory for further analysis. The soil samples collected from this investigation will be retained in our laboratory for a period of 30 days after the date of submission of the final report, after which they will be discarded unless we are notified otherwise.

Whenever possible, groundwater level observations are made during the drilling operations and are shown in the Soil Boring Logs. Prior to backfilling, each open borehole was observed again for groundwater. During drilling, the depth at which free water was observed, where drill cuttings became saturated or where saturated samples were collected, was indicated as the groundwater level during drilling. In particular, in pervious soils (granular soils), water levels are considered relatively reliable when solid or hollow-stem augers are used for drilling. However, in cohesive soils, groundwater observations are not necessarily indicative of the static water table due to low permeability rates of the soils and due to the sealing off of natural paths of groundwater during drilling operations. It should be noted that seasonal variations and recent rainfall conditions may influence the groundwater table significantly.

Soil boring logs are included in Appendix B. Also included in Appendix B are General Soil Sample Notes, and a Boring/Well Log Key that illustrates the soil classification criteria and terminology used on the Soil Boring Logs.

2.2 Laboratory Testing

Each sample recovered from the borings was examined and visually classified. This examination was performed to verify conditions identified within field boring logs, to select samples for further laboratory evaluation, and to perform visual-manual classification of samples not subject to further laboratory testing. During the examination process, the geotechnical engineer finalized the soil boring logs.

Representative soil samples were subjected to laboratory tests consisting of pocket penetrometer tests, sieve and hydrometer analysis (ASTM D422), Atterberg Limits (ASTM D4318), natural moisture content (ASTM D2216) and unconfined compression test (ASTM D2166). A brief description of each test performed by MSG is provided in Laboratory Test Procedures in Appendix C.

All soil samples were classified in general accordance with the Unified Soil Classification System (USCS). The USCS group symbol determined from the visual-manual classification is shown in parentheses at the end of the sample description for each layer shown on the Soil Boring Logs. The results of the soil classification and the laboratory test results are included on the Soil Boring Logs and Soil Laboratory Test Data, which are presented in Appendices B and C, respectively.

3.0 SUBSURFACE CONDITIONS

3.1 Subsurface Classification

The following sections describe the subsurface conditions in terms of major soil strata for the purposes of geotechnical exploration. The soil boundaries indicated are inferred from non-continuous sampling and observations of the drilling operations and/or sampling resistance. The subsurface conditions discussed in the following sections and those shown on the boring logs represent an evaluation of the subsurface conditions based on interpretation of the field and laboratory data using normally accepted geotechnical engineering judgement and common engineering practice standards. The subsurface conditions described herein may vary beyond the boring locations and at different times of the year. A generalized soil profile of the subsurface conditions encountered across the sites, beginning at the ground surface, and extended downward is as follows:

Surface Material
Approximately 8 to 9 inches of topsoil was encountered at all the soil boring locations.

Stratum 1 – Lean Clay (CL)

Stiff to hard lean clay with variable amounts of sand and gravel was encountered in all soil borings directly beneath the surface material and extended to the maximum explored depth of 25 feet below ground surface. The standard penetration number ranged from 14 to 41 and averaged 25.

3.2 Groundwater Observations

Groundwater was not encountered during and at the end of drilling operations. The lack of groundwater encountered in the borings is not necessarily an indicator of the actual water levels due to the presence of cohesive soils and their inherent property of low permeability. Typically, the level where the soil color changes from brown to gray is generally indicative of the long-term groundwater level. Notable transitions from brown to gray generally occurred at a depth of about 18.5 feet below ground surface.

Water levels reported are accurate only for the time and date the borings were drilled. The borings were backfilled and sealed the same day that they were completed. Long-term monitoring of the boreholes was not included as part of the scope of our subsurface investigation.

It should be noted that the elevation of the natural groundwater table, and the elevation and quantity of the perched groundwater, is likely to vary throughout the year depending on the amount of precipitation, runoff, evaporation and percolation in the area, as well as on the water level in the surface water bodies in the vicinity affecting the groundwater flow pattern. Long-term monitoring with monitoring wells or piezometers such is necessary to accurately assess the groundwater levels and fluctuation patterns at the site.

4.0 ANALYSES AND RECOMMENDATIONS

4.1 Preliminary Design Soil Profile

Based on our review of the subsurface soil conditions encountered during this preliminary geotechnical investigation, we have developed the following soil profile:

Table 4.1-1 Soil Profile

Stratum No	Soil Description	Depth (ft)	Total Unit Weight (pcf)	Cohesion (psf)	Friction Angle (deg)
1	Stiff to Hard Clay	0-25.0	135.0	3,000	0

4.2 Preliminary Structure Recommendations

4.2.1 SHALLOW FOUNDATIONS

Based upon our review of the existing soil conditions at the site, the subgrade soils consist of very stiff to hard lean clay at the minimum foundation bearing depth of 3.5 feet below existing grades. Provided the subgrade of the footings is properly prepared during construction, it is recommended that conventional shallow foundations bearing on the encountered native very stiff to hard clay material be designed for a net allowable bearing capacity of 4,000 psf.

By utilizing proper construction techniques, the total settlement underneath the proposed shallow foundations in the native clay soil is anticipated to be less than 1 inch and the differential settlement is expected to be ¾ of the total settlement.

The aforementioned recommended soil bearing capacity and the associated settlement estimation are based on footing elevations with regards to existing preliminary site elevations. The required footing sizes are dependent on the column and wall loads in comparison to the above recommended allowable bearing capacity of the bearing soil. However, wall footings should at least be a minimum of 18 inches wide and column footings should have a minimum dimension of 30 inches. Exterior footing bottoms and footings in unheated areas should be no less than 42 inches (typical frost depth for southern Michigan) below final exterior grade for protection against possible frost damage. Interior footings in heated buildings, which should not be subject to frost action, may bear at shallower depths, provided they are supported on native compact soil or engineered fill capable of supporting the design load.

Prior to the placement of reinforcing steel and concrete, an MSG geotechnical engineer or their designated representative should evaluate foundation excavations to verify that an adequate bearing material is present and that all debris, mud, loose, frozen or water-softened soils, and unsuitable soils are removed. All footings should bear in the undisturbed natural soils or in well-compacted engineered fill. In addition, MSG recommends that a dynamic cone penetrometer (DCP) test or Housel Penetrometer Test, or similar field testing, be performed by the geotechnical engineer representative to assure a suitable bearing capacity for all foundations prior to concrete placement.

If unsuitable materials are suspected at the subgrade elevation during construction, the project Geotechnical Engineer or their designated on-site representative should be contacted, and additional soils laboratory testing may be required. Where foundation subgrade undercutting and replacement with engineered fill is required, the undercuts should extend laterally at a slope of 1(Horizontal):2(Vertical) from the edge of the footing as shown in the typical undercutting diagram below:

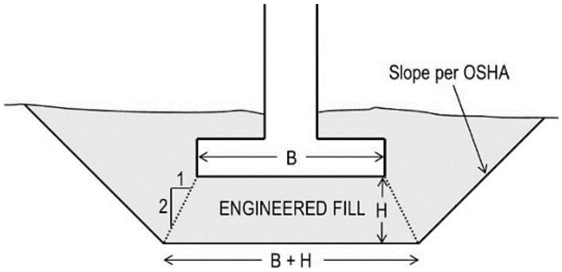


Figure 4.2.1-1 - Undercutting Diagram

Foundations should be constructed as soon as is practical after foundation excavation activities. If the foundation excavations will be left open for an extended period of time, a thin mat of lean concrete should be placed over the bottom to minimize damage to the bearing surface from weather or construction activities. Water should not be allowed to pond in any excavation. Foundation concrete should not be placed on frozen or flooded subgrade.

The final grade adjacent to the building exterior should be sloped at a minimum 2 percent grade away from the building foundations and roof drains should be routed away from the foundation soils. Foundation drains will assist in ensuring the foundation subgrade soils are not adversely impacted by moisture changes that could result in differential settlement of the foundations. To prevent moisture against the exterior footings, a perforated matted edge drain may be used around the perimeter of the footings and placed at the base of the footings. The underdrain should be backfilled with free draining material. A waterproofing membrane with a protection layer should extend from the top to the base of the footings along the exterior edge where the concrete is in direct contact with the natural or backfilled material.

If a two-pour system is used for footings and slab, the cold joint at the interface of the exterior footings and slab on grade should be located at least 4 inches above the adjacent finish exterior grade. As an alternative, the use of a waterstop between the two pours will minimize the moisture penetration through the cold joint and migration of water under the slab. A monolithic pour will eliminate the need for a waterstop.

4.2.2 SLAB-ON-GRADE

Based upon our review of the existing soil conditions at the site, an estimated modulus for subgrade reaction on top of the existing subgrade of 135 pounds per cubic inch (pci) may be used for slab-on-grade design. For a subgrade composed of well-compacted engineered fill a modulus of subgrade reaction of 175 pci may be used. The final design thickness of the floor slabs, the joint spacing and slab reinforcement should be determined by the structural engineer based on the above recommended subgrade modulus, the floor loading conditions and local building code requirements.

The subgrade of the slab-on-grade areas should be inspected and tested to assure proper preparation. The subgrade soils should be protected against frost action if construction takes place during the winter. Frozen soils should be thawed, moisture conditioned and recompacted or removed and replaced prior to commencement of slab-on-grade construction. We recommend that the floor slabs-on-grade bear directly on a minimum of 6 inches of capillary resistant granular engineered fill (well-graded granular material or engineer approved equivalent) compacted to 98 percent of Standard Proctor or 95 percent of Modified Proctor Maximum Dry Density (MDD) within 2 percent of the Optimum Moisture Content (OMC).

A waterproof membrane (vapor retarder) should be placed directly beneath the concrete to minimize infiltration of water and delamination of the concrete floor slab. The moisture condition of the floor slab should be tested prior to placement of floor coverings to verify they are within tolerable limits for the floor coverings.

In order to minimize the potential impacts caused by differential settlement, the slab-on-grade should be kept structurally separate from walls and columns and saw cut control joints should be provided at suitable intervals. A minimum of 6 inches of engineered fill should be placed between the slab bottom and the top of the footings below.

4.3 Preliminary Pavement Recommendation

Based on the soil characteristics from the preliminary geotechnical investigation, a design California Bearing Ratio (CBR) value of 5 and resilient modulus of 6,000 pci was estimated using parameters obtained from field and laboratory testing. Field CBR testing was not performed. The pavement design input parameters are established based on the procedures contained in the 1993 Guide for Design of Pavement Structures by AASHTO. For the basis of the design, MSG assumed the following input parameters:

Table 4.3-1 Assumed Pavement Design Parameters

Design Life	20 Years
Design ESAL	Light Duty: 100,000; Heavy Duty: 750,000
Reliability	80 %
Original Serviceability Index	4.2 (Flexible Paving); 4.5 (Rigid Paving)
Terminal Serviceability Index	2.0
Overall Standard Deviation	0.45 (Flexible Paving); 0.35 (Rigid Paving)

For flexible pavement design, MSG assumed structural number coefficients of 0.42 and 0.14 for asphalt concrete and aggregate base, respectively. Based on the above assumptions, recommended flexible pavement sections are provided in the following table.

Table 4.3-2 Recommended Flexible Pavement Sections

Pavement Materials	100,000 ESAL (in.)	750,000 ESAL (in.)
Surface Course	1.5	2.0
Intermediate Course	2.5	3.0
Aggregate Base	6.0	8.0

For rigid pavement design, MSG assumed a concrete elastic modulus (E_c) of 5,000,000 psi, a concrete rupture modulus (S'_c) of 700 psi and a load transfer coefficient (J) of 2.7. Based on the above assumptions, recommended rigid pavement sections are provided in the following table.

Table 4.3-3 Recommended Rigid Pavement Sections

Pavement Materials	100,000 ESAL (in.)	750,000 ESAL (in.)
Portland Cement Concrete	6.0	8.0
Aggregate Base	6.0	10.0

Site preparation recommendations presented in Section 4.4 shall be followed to provide subgrade conditions suitable for pavement support. Adequate drainage should be provided to the pavement structure to ensure a successful pavement service life is achieved. MSG highly recommends that underdrains be utilized around catch basins and in other low areas of the proposed pavements to limit the accumulation of water below the pavement structures if an asphalt pavement is preferred.

4.4 Site Preparation

Before proceeding with construction, surface soils, vegetation, topsoil, root systems, refuse, asphalt, concrete including any existing abandoned buried foundations, and other deleterious materials should be stripped from the proposed construction areas. The bearing soils should be observed by a geotechnical engineer and visually checked for suitability as a bearing soil. Depending on the time of year of construction and the Contractor's Means and Methods at controlling surface water, it may be possible that additional site subgrade material within development/construction areas will be considered unsuitable and/or unstable and will be required to be stripped during site preparation activities.

Cohesive soils are moisture sensitive and could become unstable if proper site water controls are not implemented and/or if they are subject to construction traffic. Every effort should be taken to minimize disturbance during compaction or over excavation. Where possible, free-standing water should be diverted away from the construction perimeters or pumped out using a sump to accommodate the proper compaction techniques.

Generally, areas exposed by stripping operations on which subgrade preparations are to be performed should be compacted in place to 98 percent of Standard Proctor or 95 percent of Modified Proctor MDD within 2 percent of the OMC. Soft, loose, or saturated soils that are difficult to compact may require an undercut and replacement with engineered fill for stabilization. The on-site Geotechnical Engineer or their designated representative should determine required undercut depths if necessary.

It is recommended that the prepared subgrade for pavement and slab-on-grade areas be proof-rolled to detect any unstable areas. Proof-rolling should be accomplished by making a minimum of two complete passes in each of two perpendicular directions with a fully loaded tandem-axle dump truck, or other approved pneumatic-tired vehicle, with a minimum weight of 20 tons. If proof-rolling reveals the presence of unstable areas within the subgrade, certain remedial measures will be required to stabilize the subgrade. Depending on the severity of distress encountered during proof-rolling, undercutting of 14 inches below subgrade and backfilling with engineered fill as outlined in Section 4.5 may be performed. If an undercut and replacement fails to stabilize the subgrade, use of granular backfill with geogrid stabilization may be required. Undercuts may be reduced 6 inches if geogrid and granular backfill is utilized. Alternately, chemical stabilization of the upper 14 inches with lime may be performed. It should be noted that MSG does not recommend chemical stabilization if the amount of sulfates present in the subgrade soils exceeds 5,000 parts per million (ppm). The actual undercut depths and/or subgrade remediation measures required should be determined by the on-site Geotechnical Engineer or a designated representative.

Existing utilities or underground structures in conflict with the proposed construction location should be removed and relocated or abandoned in place. If abandoned in place, it is recommended that the utility pipe be filled with cement grout to mitigate the potential for collapse in the future. Should the utility lines be removed from the site, the resultant trench excavations should be backfilled with well-compacted granular material, placed, and compacted in accordance with the recommendations of Section 4.5.

4.5 Fill Placement and Engineered Fill Requirements

All new fill should consist of inorganic soil that is free from all deleterious materials and construction debris. Fill materials should not be placed in a frozen condition or upon frozen subgrades. Proper drainage should be maintained during and after fill placement to prevent water from impacting compaction efforts or long-term fill integrity. All fine-grained fill soils should be checked for plasticity index and liquid limit before placement. Cohesive fill materials should have a liquid limit less than 40 percent and plasticity index less than 20 percent (i.e., non-expansive). The on-site cohesive soils should be evaluated prior to reuse as structural fill by a qualified geotechnical engineer.

Coarse crushed granular material is recommended as fill for utility trench backfill, undercut areas, and as aggregate base material for pavement and slab-on-grade areas. The granular material shall consist of natural aggregate materials that meet the gradation requirements of MDOT 21AA or engineer approved equivalent. Typical lift thickness utilized for this material is 8 inches. In utility trenches, granular backfill material should extend at least two pipe diameters above the pipe's crown. As an alternative to imported granular fill, excavated soil material may be recompacted back in place so long as the excavated soil material is determined to be suitable. If a working platform for construction is needed, and prior to footing excavation, it is recommended that at least 6 inches of granular base material meeting the gradation requirements MDOT 21AA aggregate.

Fill should be compacted to 98 percent of the Standard Proctor or 95 percent of Modified Proctor MDD and should be compacted within 2 percent of OMC. Fill materials should be placed in horizontal lifts and adequately keyed into stripped and scarified subgrade soils and adjacent fill. A qualified geotechnical consultant should be retained to monitor fill placement in order to assure compaction requirements are achieved. Soil density testing should be performed during fill placement activities to assure proper fill compaction. A commonly used testing criterion is one test per 2,500 square feet per lift in areas to support proposed structures and one test per 5,000 square feet in parking lots, driveways, exterior slabs, etc., with a minimum of three tests per lift. Areas that do not achieve compaction requirements after initial placement should be recompacted to meet project requirements.

The actual lift thickness suitable for fill placement is dependent upon the soil type, compaction equipment, and the compaction specification. In general, fill should be placed in a 9-inch loose lift thickness (8-inch compacted); assuming appropriately weighted and ballasted compaction equipment is utilized. In confined areas where hand operated compaction equipment is required, 4-inch and 6-inch loose lift thickness should be utilized for hand operated vibratory plate compactors and hand operated vibratory drum rollers weighing at least 1,000 pounds, respectively. Sand fills should be compacted using smooth vibratory rollers. Clay fills should be compacted using a sheep foot compactor. The geotechnical engineer, as part of the construction monitoring, should review the equipment utilized for compaction to confirm suitability relative to the specified loose lift thickness. If necessary, the geotechnical engineer will recommend a revised lift thickness suitable to the equipment performing compaction.

To minimize corrosion of existing metallic utilities, topsoil, organic soils, existing fill soils, and mixtures of sand and clay should not be placed adjacent to metallic utilities. In addition, buried utilities of different metallic materials should be electrically isolated from each other to minimize galvanic corrosion.

4.6 Lateral Earth Pressures

Lateral earth pressures (horizontal stresses) are developed during soil displacements (strains). Lateral earth pressure for design is determined utilizing an earth pressure coefficient to relate horizontal stress to vertical stress. Three separate earth pressure coefficients are used to determine lateral earth pressure: at-rest; active; and passive.

Applied horizontal stress can be determined by multiplying the appropriate earth pressure coefficient by the applied vertical stress. Earth pressure coefficients are a direct function of the internal friction of a soil. Laboratory testing to determine internal friction angles for soil was not performed. However, index laboratory and field data obtained can be utilized to approximate earth pressure coefficients based upon empirical relationships. Lateral earth pressure coefficients for soils encountered during this investigation are provided in Table 4.6-1.

Table 4.6-1 Recommended Lateral Earth Parameters

Soil Parameters	Engineered Granular Soil	Existing Soils
		Lean Clay (Stratum 1)
Total Unit Weight (pcf)	125	135
Internal Friction Angle (°)	30.0	25.0
At-rest Pressure Coefficient, K_o	0.50	0.58
Active Pressure Coefficient, K_a	0.33	0.40
Passive Pressure Coefficient, K_p	3.0	2.5
Concrete/Soil Friction Coefficient	0.5	0.0
Concrete/Soil Adhesion Factor	0.0	0.2

For retaining walls, to minimize lateral earth pressures, MSG recommends the zone adjacent to any walls be backfilled with granular fill. To provide effective drainage, a zone of free-draining gravel (similar to MDOT 6AA gravel) should be used directly adjacent to the walls with a minimum thickness of 18 inches. This granular zone should drain to weepholes or a pipe drainage system to prevent hydrostatic pressures from developing against the walls.

The type of backfill beyond the free-draining granular zone will govern the magnitude of the pressure to be used for structural design. Clean granular soil is recommended as the backfill material against retaining structures to minimize lateral earth pressure. Lateral earth pressure coefficients for engineered fill are provided in Table 4.6-1.

The coefficients of friction between concrete and soil subgrade were also provided in the table above. These coefficients can be used for evaluating the factor of safety against sliding of foundations. The recommended minimum safety factor against sliding is 1.5. Passive pressure resistance of the top 3 feet below final grade should generally be neglected in designing the retaining walls to resist sliding failure due to the freeze-thaw cycle that can significantly weaken soils and the potential for the material to be removed at a future date for installation of utilities or other construction-related activities.

Any additional lateral earth pressure due to surcharge loading conditions including, but not limited to, floor loads, column loads, sloping backfill, traffic loading, and construction loads, should be incorporated into the wall design. MSG should be retained to perform other detailed geotechnical evaluations for retaining walls, as necessary, including but not limited to, settlement and global stability. A detailed geotechnical evaluation and structural design of retaining walls is beyond the scope of this report.

4.7 Site Seismic Classification

According to ASCE 7-22 Table 20.2-1, the proposed site is designated as "Site Class CD" based on the average number of the Standard Penetration Test N values and soil shear strength for the upper 25 feet of soil (the maximum depth the borings were advanced for this investigation) and assumed subsurface conditions to a depth of 100 feet.

5.0 CONSTRUCTION CONSIDERATIONS

5.1 Groundwater Control

Groundwater was not encountered during or at the end of drilling operations. However, the long-term groundwater level may be estimated at 18.5 feet below existing grade based on notable color transitions of the cohesive soil from brown to gray, which may indicate the presence of a long-term groundwater table.

The location of the level of groundwater is of importance in shallow foundations for a number of reasons. Most importantly, the bearing capacity of the soil is affected by the presence of a high-water table, decreasing the bearing capacity. Typically, the groundwater elevation fluctuates and is higher during the winter and spring and lower in summer and early fall. It should be noted that groundwater seepage will have a significant impact on construction activities.

For standard shallow excavations, significant problems associated with groundwater seepage into the excavation are not anticipated, provided the Contractor be prepared to address localized groundwater accumulation, perched groundwater, or surface run off (i.e., pumping water from prepared sumps). The amount and type of dewatering required during construction will be further impacted by the weather, groundwater levels at the time of construction, the effectiveness of the Contractor's techniques in preventing surface water runoff from entering open excavations, and their ability to lower the groundwater table. The final design of any temporary earth support structures for excavations, as well as the associated dewatering and groundwater control plan, will be completed by the Contractor.

To manage surface water, slopes in pavement areas should consist of 1.5 percent slopes towards inlets or drainage structures, building exteriors should have a minimum of 2 percent slopes away from the building and discharge points (i.e., roof down spouts) should consist of closed conduits and divert away from the buildings to inlets or drainage structures. The use of perimeter drains and/or sub-drains may be necessary on approval of the site civil design engineer. The project civil engineer is responsible for designing the surface drainage improvements.

5.2 Excavations and Slope

Familiarity with applicable local, state, and federal safety regulations, including current OSHA excavation and trench safety is vital. Therefore, it should be a requisite for both the Owner and Contractor with the Contractor by and large being responsible for the safety of the site. Activities at the site, such as utilities or building demolition and site preparation, may require excavations at significant depths below the ground surface. Slope height, slope inclination, and excavation depth (including utility trench excavations) should in no case exceed those specified in local, state, or federal safety (OSHA Health and Safety Standards for Excavations, 29 CFR Part 1926 Subpart P) regulations. Such regulations are strictly enforced and, if not followed, the Owner, Contractor, or earthwork or utility Subcontractors could be liable for substantial penalties.

The overburden soils encountered during our investigation were generally composed of stiff to hard clay. Based upon the data obtained, we anticipate OSHA will classify the native clay soils as **Type B** soil, which will be limited to a maximum temporary excavation slope of 1(H):1(V). Flatter slopes are required where soils are stockpiled or in the vicinity of existing structures. If sufficient room is not available for sloping the excavation walls, temporary shoring will be required. It is our recommendation that any excavation in excess of 5 feet in depth, should be designed by a professional engineer.

6.0 GENERAL QUALIFICATIONS AND LIMITATIONS

The evaluations, conclusions and recommendations in this report are based on our interpretation of the field and laboratory data obtained during the preliminary geotechnical investigation, our understanding of the project and our experience during previous work, with similar sites and subsurface conditions. Data used during this limited exploration included:

- Four (4) soil borings performed during this preliminary investigation.
- Observations of the project site by our staff;
- Results of laboratory soil testing; and,
- Results of the preliminary geotechnical analyses.

The subsurface conditions discussed in this report and those shown on the boring logs represent an estimate of the subsurface conditions based on interpretation of the boring data using normally accepted geotechnical engineering judgments. 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. MSG is not responsible for independent conclusions, opinions, or recommendations made by others based upon information presented in this report.

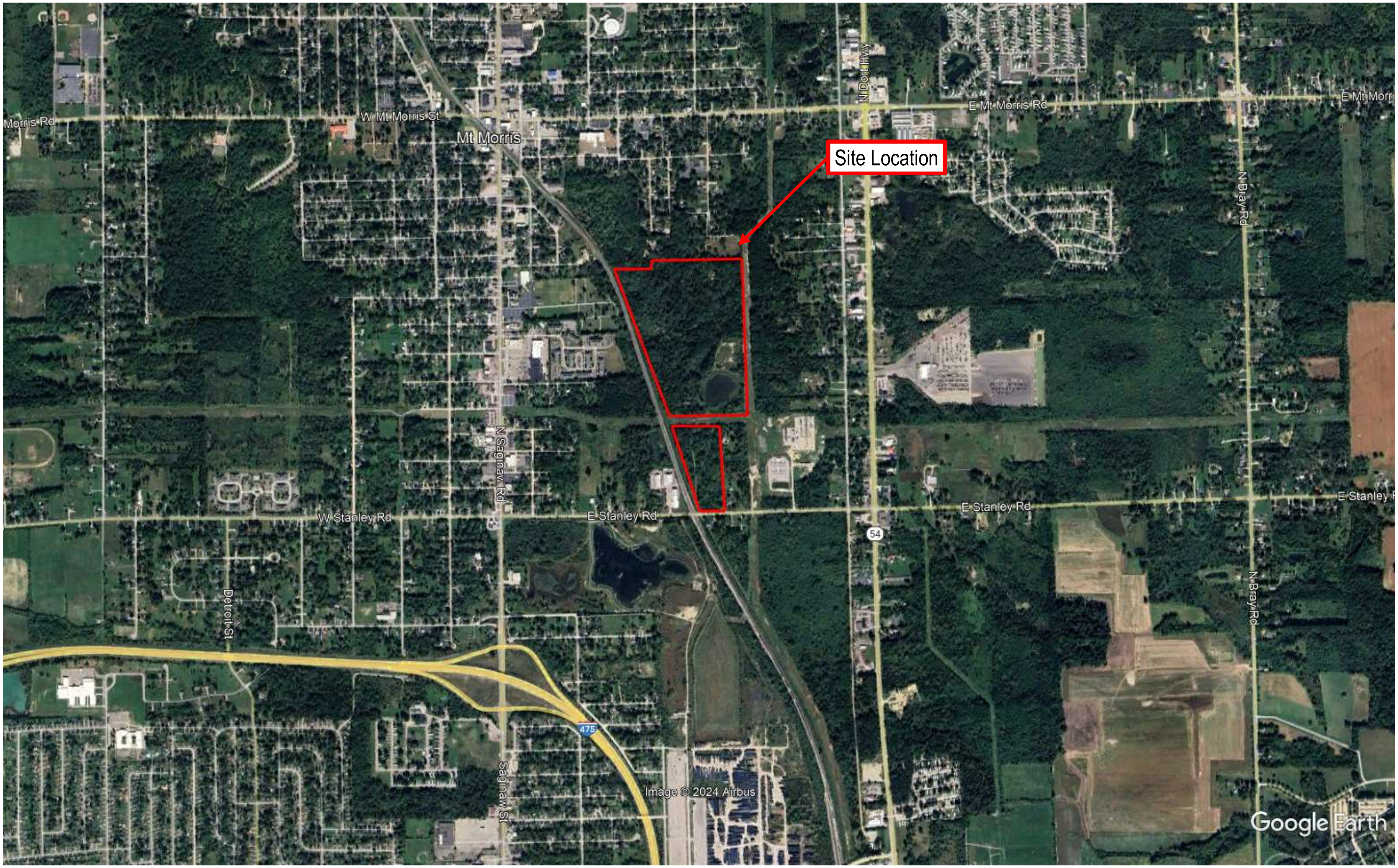
We strongly recommend the final project plans and specifications be reviewed by MSG’s geotechnical engineer to confirm that the geotechnical aspects are consistent with the recommendations of this report. In particular, the specifications for excavation and foundation construction should be prepared and/or reviewed by MSG’s Geotechnical Engineer of Record. In addition, we recommend site subgrade preparation, fill compaction activities, and foundation installation activities should be monitored by MSG’s geotechnical engineer or his/her representative.

This report and evaluation reflect the geotechnical aspects of the subsurface conditions at the site. Review and evaluation of environmental aspects of subsurface conditions are beyond the scope of this report.

APPENDIX A

FIGURE 1 – SITE LOCATION MAP

FIGURE 2 – SOIL BORING LOCATION MAP



Mannik Smith GROUP

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Figure 1: Site Location Map
DRP RACER Genesee Industrial Land
Genesee Township, Michigan
MSG Project Number: 401.2401267.000

No Scale
Map Adapted from
Google Earth 2024 ®

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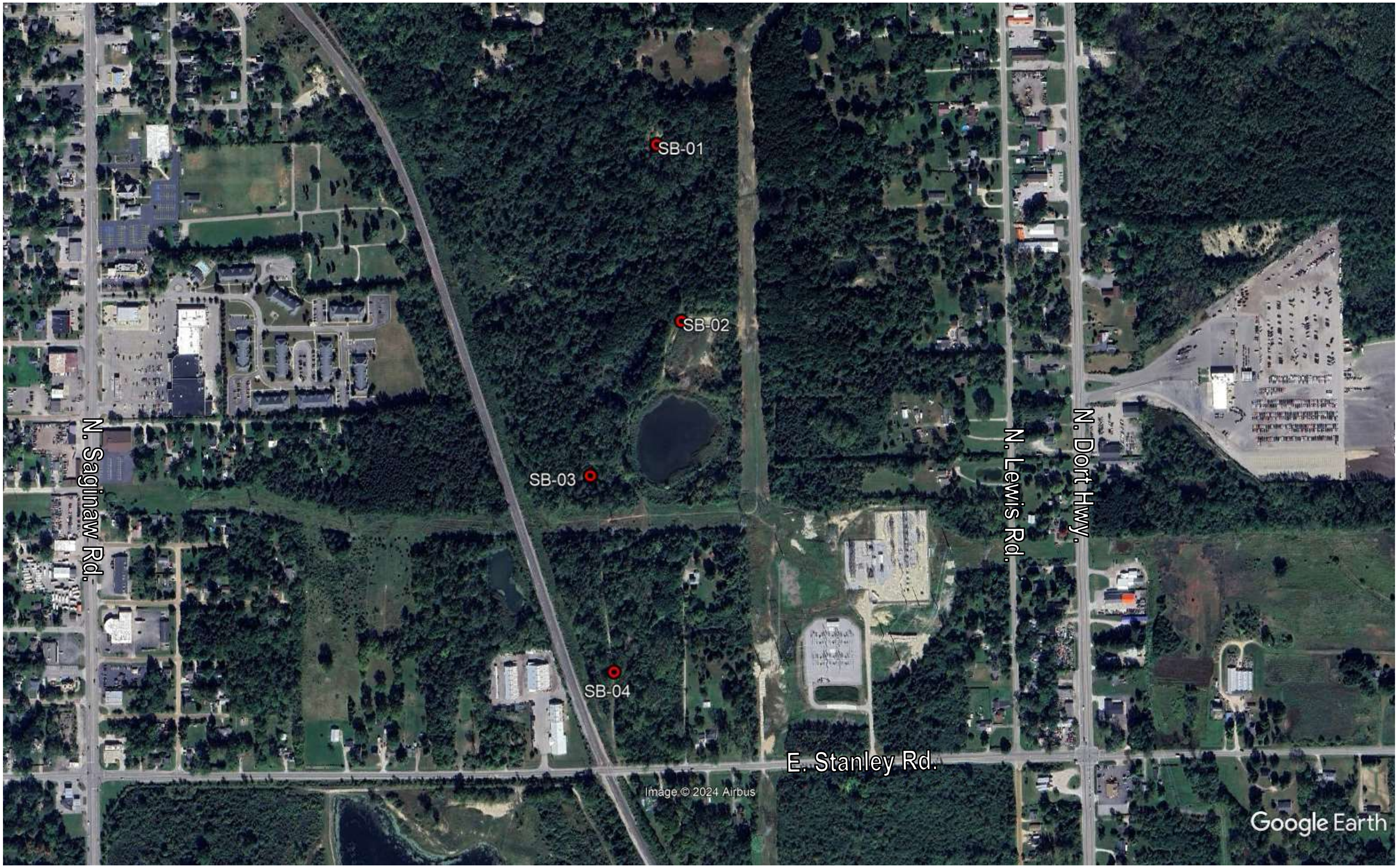

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Fax: 734-397-3131
www.MannikSmithGroup.com

Figure 2: Soil Boring Location Map
DRP RACER Genesee Industrial Land
Genesee Township, Michigan
MSG Project Number: 401.2401267.000

No Scale
Map Adapted from
Google Earth 2024 ®

N

APPENDIX B

SOIL BORING LOGS

GENERAL SOIL SAMPLE NOTES

Unless noted, all terms utilized herein refer to the Standard Definitions presented in ASTM D653.

Standard Penetration Test (ASTM D1586): A 2.0-inch outside-diameter (O.D.), 1-3/8-inch inside-diameter (I.D.) split barrel sampler is driven into undisturbed soil by means of a 140-pound weight falling freely through a vertical distance of 30 inches. The sampler is normally driven three successive 6-inch increments. The total number of blows required for the final 12 inches of penetration is the Standard Penetration Resistance (N).

COHESIVE SOILS			COHESIONLESS SOILS	
Consistency	Approximate Range of N	Unconfined Compressive Strength (psf)	Density Classification	Approximate Range of N
Very Soft	0 – 1	Below 500	Very Loose	0 – 4
Soft	2 – 4	500 – 1,000	Loose	5 – 10
Medium Stiff	5 – 8	1,000 – 2,000	Medium Dense	11 – 30
Stiff	9 – 15	2,000 – 4,000	Dense	31 – 50
Very Stiff	16 – 30	4,000 – 8,000	Very Dense	Over 50
Hard	31 – 50	8,000 – 16,000		
Very Hard	Over 50	Over 16,000		

CLASSIFICATION

The major soil constituent is the principal noun, i.e. sand, silt, gravel. The second major soil constituent and other minor constituents are reported as follows:

Second Major Constituent (percent by weight)	Minor Constituents (percent by weight)
Trace – 1% to 11%	Trace – 1% to 11%
Adjective – 12% to 35% (clayey, silty, etc.)	Little – 12% to 22%
	Some – 23% to 33%
And – Over 35%	

PARTICLE SIZES

Boulders		- Greater than 12 inches (305 mm)
Cobbles		- 3 inches (76.2 mm) to 12 inches (305 mm)
Gravel:	Coarse	- ¾ inches (19.05 mm) to 3 inches (76.2 mm)
	Fine	- No.4 (4.75 mm) to ¾ inches (19.05 mm)
Sand:	Coarse	- No. 10 (2.00 mm) to No. 4 (4.75 mm)
	Medium	- No. 40 (0.425 mm) to No. 10 (2.00 mm)
	Fine	- No. 200 (0.074 mm) to No. 40 (0.425 mm)
Silt		- 0.005 mm to 0.074 mm
Clay		- Less than 0.005 mm

If clay content is sufficient so that clay dominates soil properties, clay becomes the principal noun with the other major soil constituent as modifier: i.e., silty clay. Other minor soil constituents may be included in accordance with the classification breakdown for cohesionless soils: i.e., silty clay, trace sand, little gravel.

If sand particle size is greater than 11% by weight of the total sample weight, the adjective (i.e., fine, medium or coarse) is added to the soil description for the sand portion of the sample, provided sand is the major or second major constituent.

SAMPLE DESIGNATIONS

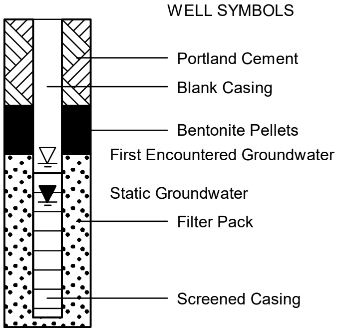
AS	Auger Sample - directly from auger flight	ST	Shelby Tube Sample - 3-inch diameter unless otherwise noted
BS	Miscellaneous Samples - Bottle or Bag	PS	Piston Sample - 3-inch diameter unless otherwise noted
MC	Macro-Core Sample - 2.25-inch O.D., 1.75-inch I.D., 5 feet long polyethylene liner	RC	Rock Core - NX core unless otherwise noted
LB	Large-Bore (Micro-Core) Sample - 1-inch diameter, 2 feet long polyethylene liner	CS	CME Continuous Sample - 5 feet long, 3-inch diameter unless otherwise noted
SS	Split Spoon Sample - 1-inch or 2-inch O.D.	HA	Hand Auger
LS	Split Spoon (SS) Sampler with 3 feet long liner insert	DP	Drive Point
NR	No Recovery	CM	Coring Machine

MAJOR DIVISIONS					TYPICAL NAMES
COARSE-GRAINED SOILS MORE THAN HALF IS COARSER THAN NO. 200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS WITH LESS THAN 15% FINES	GW		WELL-GRADED GRAVELS WITH OR WITHOUT SAND
			GP		POORLY-GRADED GRAVELS WITH OR WITHOUT SAND
		GRAVELS WITH 15% OR MORE FINES	GM		SILTY GRAVELS WITH OR WITHOUT SAND
			GC		CLAYEY GRAVELS WITH OR WITHOUT SAND
	SANDS MORE THAN HALF COARSE FRACTION IS FINER THAN NO. 4 SIEVE SIZE	CLEAN SANDS WITH LESS THAN 15% FINES	SW		WELL-GRADED SANDS WITH OR WITHOUT GRAVEL
			SP		POORLY-GRADED SANDS WITH OR WITHOUT GRAVEL
		SANDS WITH 15% OR MORE FINES	SM		SILTY SANDS WITH OR WITHOUT GRAVEL
			SC		CLAYEY SANDS WITH OR WITHOUT GRAVEL
FINE-GRAINED SOILS MORE THAN HALF IS FINER THAN NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT 50% OR LESS		ML		INORGANIC SILTS OF LOW TO MEDIUM PLASTICITY WITH OR WITHOUT SAND OR GRAVEL
			CL		INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY WITH OR WITHOUT SAND OR GRAVEL
			OL		ORGANIC SILTS OR CLAYS OF LOW TO MEDIUM PLASTICITY WITH OR WITHOUT SAND OR GRAVEL
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50%		MH		INORGANIC SILTS OF HIGH PLASTICITY WITH OR WITHOUT SAND OR GRAVEL
			CH		INORGANIC CLAYS OF HIGH PLASTICITY WITH OR WITHOUT SAND OR GRAVEL
			OH		ORGANIC SILTS OR CLAYS OF HIGH PLASTICITY WITH OR WITHOUT SAND OR GRAVEL
HIGHLY ORGANIC SOILS			PT		PEAT AND OTHER HIGHLY ORGANIC SOILS

SYMBOLS KEY

SAMPLE TYPES

- Grab Sample
- Rock Core
- Split Spoon sample, 1 inch or 2 inch outer-diameter.
- ■ Shelby Tube sample - 3 inch diameter unless otherwise noted.



OTHER MATERIAL SYMBOLS

- Topsoil
- Poorly Graded Sand with Clay
- Clayey Sand
- Sandy Silt
- Gravelly Silt
- Poorly Graded Gravelly Sand
- Well Graded Gravel with Clay
- Well Graded Gravel with Silt
- Well Graded Gravelly Sand
- Shale
- Shaly Dolomite
- Limestone

BORING / WELL LOG KEY

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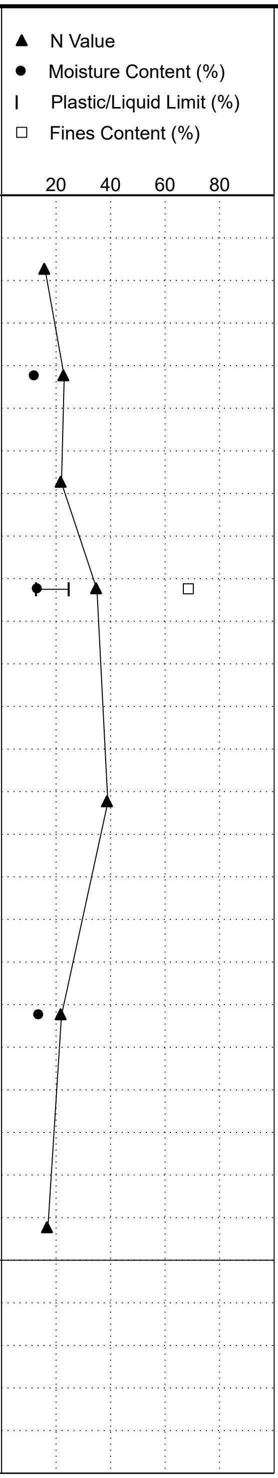
BOREHOLE NUMBER SB-01

Sheet 1 of 1

CLIENT Detroit Regional Partnership
PROJECT NAME DRP RACER Genesee Industrial Land
PROJECT NUMBER 401.2401267.000
PROJECT LOCATION Genesee Township, Michigan
DATE STARTED 12-12-2024 **COMPLETED** 12-12-2024
POSITION N: 588755± ft E: 13305701± ft (NAD 1983 Michigan South (Intl Feet))
DRILLING CONTRACTOR The Mannik & Smith Group, Inc.
SURFACE ELEVATION 787± ft **FINAL DEPTH** 25.0 ft
DRILLING METHOD Direct Push
LOGGED BY MH **CHECKED BY** GVA
EQUIPMENT Geoprobe 3230DT **Operator** RJS
REMARKS Elevation estimated from Google Earth™.

DEPTH (ft)	ELEVATION (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)
0.7	786.3		TOPSOIL - 8 INCHES				
			Sandy Lean CLAY (CL): very stiff; brown; damp.	SPT SS-1	56	5-7-9 (16)	3.00
				SPT SS-2	61	6-11-12 (23)	4.00
5	782						
				SPT SS-3	56	5-9-13 (22)	3.50
8.5	778.5		Lean CLAY with sand (CL): hard; brown; moist.	SPT SS-4	61	10-15-20 (35)	4.50
10	777						
				SPT SS-5	72	15-18-21 (39)	4.00
15	772						
18.5	768.5		Sandy Lean CLAY (CL): stiff to very stiff; gray; moist.	SPT SS-6	78	7-9-13 (22)	1.50
20	767						
				SPT SS-7	56	6-8-9 (17)	2.00
25.0	762.0		Terminated at 25.00 ft. Reached Target Depth.				



LEGEND:

∇ AT TIME OF DRILLING ____________
▼ AT END OF DRILLING ____________
∇ AFTER DRILLING ____________

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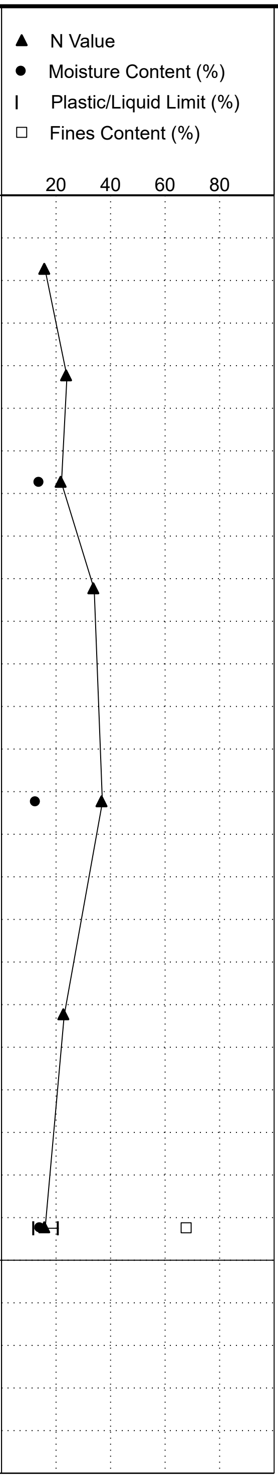
BOREHOLE NUMBER SB-02

Sheet 1 of 1

CLIENT Detroit Regional Partnership
PROJECT NUMBER 401.2401267.000
DATE STARTED 12-12-2024 **COMPLETED** 12-12-2024
DRILLING CONTRACTOR The Mannik & Smith Group, Inc.
DRILLING METHOD Direct Push
EQUIPMENT Geoprobe 3230DT **Operator** RJS

PROJECT NAME DRP RACER Genesee Industrial Land
PROJECT LOCATION Genesee Township, Michigan
POSITION N: 587829± ft E: 13305796± ft (NAD 1983 Michigan South (Intl Feet))
SURFACE ELEVATION 786± ft **FINAL DEPTH** 25.0 ft
LOGGED BY MH **CHECKED BY** GVA
REMARKS Elevation estimated from Google Earth™.

DEPTH (ft)	ELEVATION (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)
		TOPSOIL - 9 INCHES				
0.8	785.2	Sandy Lean CLAY (CL): very stiff; brown; damp.				
			SPT SS-1	89	6-6-10 (16)	3.00
			SPT SS-2	61	7-12-12 (24)	3.50
5	781					
			SPT SS-3	78	6-8-14 (22)	2.50
8.5	777.5	Lean CLAY with sand (CL): hard; brown; moist.				
			SPT SS-4	56	11-14-20 (34)	4.50
10	776					
			SPT SS-5	50	16-17-20 (37)	4.00
15	771					
18.5	767.5	Sandy Lean CLAY (CL): very stiff; gray; damp.				
			SPT SS-6	67	8-8-15 (23)	4.00
20	766					
			SPT SS-7	56	7-7-9 (16)	2.00
25.0	761.0	Terminated at 25.00 ft. Reached Target Depth.				



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▽ AT TIME OF DRILLING ____________
▼ AT END OF DRILLING ____________
▽ AFTER DRILLING ____________

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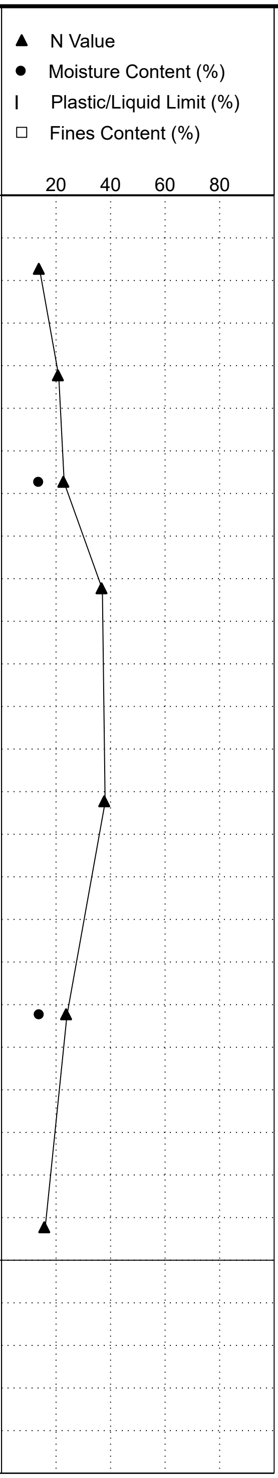
BOREHOLE NUMBER SB-03

Sheet 1 of 1

CLIENT Detroit Regional Partnership
PROJECT NUMBER 401.2401267.000
DATE STARTED 12-12-2024 **COMPLETED** 12-12-2024
DRILLING CONTRACTOR The Mannik & Smith Group, Inc.
DRILLING METHOD Direct Push
EQUIPMENT Geoprobe 3230DT **Operator** RJS

PROJECT NAME DRP RACER Genesee Industrial Land
PROJECT LOCATION Genesee Township, Michigan
POSITION N: 587098± ft E: 13305381± ft (NAD 1983 Michigan South (Intl Feet))
SURFACE ELEVATION 789± ft **FINAL DEPTH** 25.0 ft
LOGGED BY MH **CHECKED BY** GVA
REMARKS Elevation estimated from Google Earth™.

DEPTH (ft)	ELEVATION (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)
0.7	788.3	TOPSOIL - 8 INCHES				
		Sandy Lean CLAY (CL): stiff to very stiff; brown; damp.	SPT SS-1	50	7-5-9 (14)	3.00
			SPT SS-2	56	8-9-12 (21)	4.00
5	784					
		- Qu = 5,550 psf	SPT SS-3	67	6-8-15 (23)	3.50
8.5	780.5	Lean CLAY with sand (CL): hard; brown; moist.	SPT SS-4	56	9-16-21 (37)	4.00
10	779					
			SPT SS-5	72	16-19-19 (38)	4.00
15	774					
18.5	770.5	Sandy Lean CLAY (CL): very stiff; gray; damp.	SPT SS-6	89	8-8-16 (24)	3.00
20	769					
			SPT SS-7	50	8-8-8 (16)	2.00
25.0	764.0	Terminated at 25.00 ft. Reached Target Depth.				



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▼ AT END OF DRILLING
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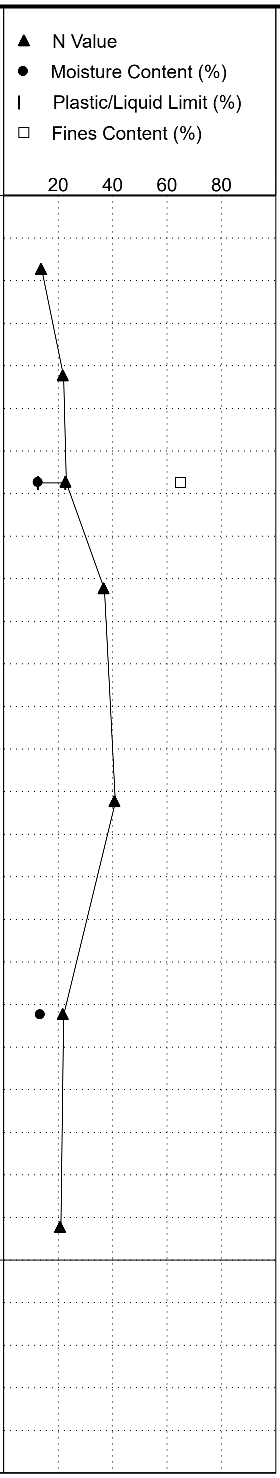
BOREHOLE NUMBER SB-04

Sheet 1 of 1

CLIENT Detroit Regional Partnership
PROJECT NUMBER 401.2401267.000
DATE STARTED 12-12-2024 **COMPLETED** 12-12-2024
DRILLING CONTRACTOR The Mannik & Smith Group, Inc.
DRILLING METHOD Direct Push
EQUIPMENT Geoprobe 3230DT **Operator** RJS

PROJECT NAME DRP RACER Genesee Industrial Land
PROJECT LOCATION Genesee Township, Michigan
POSITION N: 586119± ft E: 13305508± ft (NAD 1983 Michigan South (Intl Feet))
SURFACE ELEVATION 794± ft **FINAL DEPTH** 25.0 ft
LOGGED BY MH **CHECKED BY** GVA
REMARKS Elevation estimated from Google Earth™.

DEPTH (ft)	ELEVATION (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)
		TOPSOIL - 9 INCHES				
0.8	793.2	Sandy Lean CLAY (CL): stiff to very stiff; brown; damp.	SPT SS-1	56	6-6-8 (14)	3.00
			SPT SS-2	50	9-9-13 (22)	3.50
5	789					
		- Qu = 7,183 psf	SPT SS-3	78	7-7-16 (23)	3.50
8.5	785.5	Lean CLAY with sand (CL): very stiff to hard; brown; moist.	SPT SS-4	72	16-15-22 (37)	3.50
10	784					
			SPT SS-5	67	15-20-21 (41)	4.00
15	779					
18.5	775.5	Sandy Lean CLAY (CL): very stiff; gray; damp.	SPT SS-6	56	9-7-15 (22)	2.00
20	774					
			SPT SS-7	67	9-10-11 (21)	2.50
25.0	769.0	Terminated at 25.00 ft. Reached Target Depth.				



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APPENDIX C

SOIL LABORATORY TEST DATA

LABORATORY TEST PROCEDURES

A brief description of the most common laboratory tests performed at the Geotechnical Engineering Laboratory at the Mannik Smith Group is provided in the following sections.

DESCRIPTION OF SOILS (VISUAL-MANUAL PROCEDURE) (ASTM D2488)

The visual classification of soil samples are performed in accordance with ASTM D2488 standard. Our engineers use this test method to describe each soil sample using visual examination and simple manual tests. Visual classification helps grouping similar soil samples so that only a minimum number of laboratory tests are required for positive soil classification.

POCKET PENETROMETER

In the pocket penetrometer test, the unconfined compressive strength of a cohesive soil sample is estimated by measuring the resistance of the sample to the penetration of a small, calibrated spring-loaded cylinder. The maximum capacity of the penetrometer is 4.5 tons per square foot.

NATURAL MOISTURE CONTENT (ASTM D2216)

Natural moisture content represents the ratio of the weight of water in a given amount of soil to the weight of solid particles. Natural moisture content is expressed as a percentage (%). In this test method the water content is measured in the laboratory by noting the weight loss after drying the soil at specific temperature for 24 hours.

ATTERBERG LIMITS (ASTM D4318)

The Atterberg Limits test is performed in accordance with ASTM D4318. Liquid Limit (LL), Plastic Limit (PL) and Plasticity Index (PI) of the soil sample are determined using this test method. The Liquid Limit is the moisture content at which the soil begins to behave as a liquid material and starts to flow. The Plastic Limit is the moisture content at which the soil changes from plastic to semi-solid stage. The Plasticity Index (PI = LL - PL) is the range of moisture content at which the soil is in a plastic stage. Typically, a soil's potential for volume change increases with increase of plasticity indices.

PARTICLE SIZE ANALYSIS (ASTM D421, D422 and D1140)

These tests are performed to determine the partial soil particle size distribution. The soil sample is prepared according to ASTM D421 test method. The amount of material finer than the openings on the No. 200 sieve (0.075 mm) is determined by wash sieve method according to ASTM D1140. The hydrometer test is used to determine particle size distribution of material finer than 0.075 mm according to ASTM D422 test method.

STANDARD PROCTOR COMPACTION TEST (ASTM D698)

The Standard Proctor compaction test is used to determine maximum dry density and optimum moisture content of the soil sample. In this test, the soil is compacted in the Proctor mold in three lifts of equal volume using a standard effort by the free falling of a 5.5 lb rammer from 12 inches above soil surface. The test procedure is repeated on samples at several different moisture contents and a parabolic graph showing the relationship between moisture content and dry density of the soil is established. The maximum dry unit weight of the compacted sample and the respective moisture content is reported as maximum dry density and optimum moisture content of the soil sample.

MODIFIED PROCTOR COMPACTION TEST (ASTM D1557)

Modified Proctor compaction is similar to the Standard Proctor test. In this test, the soil is compacted in the Proctor mold in five lifts of equal volume using a standard effort by the free falling of a 10 lb rammer from 18 inches above the soil surface. The maximum dry unit weight of the compacted sample and the respective moisture content is reported as maximum dry density and optimum moisture content of the soil sample.

LABORATORY CALIFORNIA BEARING RATIO (ASTM D1883)

The CBR value is the ratio of forces required for 0.1-inch penetration of a 2-inch diameter circular plunger at the rate of 0.05 inch/min into a compacted soil sample compared to the same penetration in a certain standard crushed stone.

LOSS ON IGNITION TEST (LOI) (ASTM D2974)

LOI tests are performed on peat or suspected organic soils. An oven-dried sample is ignited in a furnace at 440°C (Method C) or 750°C (Method D). The ash content of the soil sample is determined as a percentage of the weight of the oven-dried sample. The organic content is the loss of weight due to ignition and reported as a percentage of the weight of the oven-dried sample.

ONE-DIMENSIONAL CONSOLIDATION TEST (ASTM D2435)

The consolidation test data is used to estimate the magnitude and rate of both differential and total settlement of a structure. A one-dimensional consolidation test is performed in a consolidation ring that does not allow lateral displacement of the sample. The sample is subjected to various vertical loading and unloading cycles. The deformation of the sample due to loading and unloading is recorded and used for the plotting a void ratio-applied pressure graph. The pre-consolidation pressure for the soil can also be determined from this test.

UNCONFINED COMPRESSION TEST ON ROCK SAMPLES (ASTM D7012)

In the unconfined compression test, the unconfined compressive strength (q_u) of a rock sample is estimated by measuring the resistance of the sample in compression when an axial loading is applied to the cylindrical specimen (with a height to diameter ratio of approximately 2) to reach the failure condition.

UNCONFINED COMPRESSION TEST ON SOIL SAMPLES (ASTM D2166)

In the unconfined compression test, the unconfined compressive strength (q_u) of a cohesive soil sample is estimated by measuring the resistance of the sample in compression when an axial loading is applied to the cylindrical specimen (with a height to diameter ratio of 2 to 2.5) to reach the failure condition or 15 percent (%) of axial deformation, whichever is secured first.

UNCONSOLIDATED-UNDRAINED (UU) TRIAXIAL COMPRESSION TEST (ASTM D2850)

Triaxial Shear tests are used to determine the shear strength of soil samples under various loading conditions. The test is performed on a relatively undisturbed sample extruded from a Shelby tube. In this test method, fluid flow is not permitted into or out of the soil specimen as the load is applied (undrained condition), therefore pore pressure builds up in the sample. The compressive strength of a soil is determined in terms of the total stress. The various confining pressures help determining the shear strength of the soil at different depths.

CONSOLIDATED-UNDRAINED (CU) TRIAXIAL COMPRESSION TEST (ASTM D4767)

The shear characteristics of cohesive samples (collected from relatively undisturbed sample extruded from a Shelby tube) are measured in this test under undrained conditions. This test represents field conditions where fully consolidated soils under one set of stresses are subjected to a sudden change in stress without sufficient time for further consolidation (undrained condition). The data from this test is used to analyze the shear strength parameters of the soil at different depths. The compressive strength of a soil is reported in terms of the effective stress.

WATER SOLUBLE SULFATE, RESISTIVITY AND PH

To evaluate the corrosion potential of the site, MSG performs sulfates (Ohio DOT Supplement 1122), resistivity (ASTM G187), and pH tests (ASTM D4972) on select soil samples.

SPECIFIC GRAVITY (ASTM D854)

Specific gravity is defined as the ratio of the unit weight of soil solids only to unit weight of water at a specific temperature. MSG performs specific gravity tests for soils according to ASTM D854 test procedure.

PERMEABILITY (ASTM D2434 and ASTM D5084)

This test method covers laboratory measurements of the hydraulic conductivity (the coefficient of permeability) of water-saturated granular and cohesive materials. MSG performs multiple methods for permeability tests according to ASTM D2434 and ASTM D5084.

DIRECT SHEAR TEST (ASTM D3080)

The direct shear tests are performed to determine the maximum and residual shear strength. A horizontal load is applied at a constant rate of strain. The soil sample is placed in a box where the lower half of the box is mounted on rollers and is pushed forward at a uniform rate by a motorized apparatus. The upper half of the box bears against a steel proving ring, the deformation of which is shown on a dial gauge indicating the shear force. The various information that can be obtained from the results includes the maximum (peak) shear strength and the ultimate (residual) shear strength.

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SUMMARY OF LABORATORY RESULTS

AASHTO R18

CLIENT Detroit Regional Partnership

PROJECT NAME DRP RACER Genesee Industrial Land

PROJECT NUMBER 401.2401267.000

PROJECT LOCATION Genesee Township, Michigan

Boring No. / Sample No.	Depth	Liquid Limit	Plastic Limit	Plasticity Index	Maximum Size (mm)	%<#200 Sieve	Class-ification	Water Content (%)	Bulk Density (pcf)	Satur-ation (%)	Specific Gravity
SB-01 / SS-2	3.5							12.2			
SB-01 / SS-4	8.5	25	13	12	9.525	69	CL	13.3			
SB-01 / SS-6	18.5							13.8			
SB-02 / SS-3	6.0							13.9			
SB-02 / SS-5	13.5							12.6			
SB-02 / SS-7	23.5	21	12	9	9.525	68	CL	14.2			
SB-03 / SS-3	6.0							13.8	132.8		
SB-03 / SS-6	18.5							14.0			
SB-04 / SS-3	6.0	23	13	10	19	65	CL	12.8	141.2		
SB-04 / SS-6	18.5							13.6			

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ph: (734) 397-3100 fax: (734) 397-3131
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GRAIN SIZE DISTRIBUTION

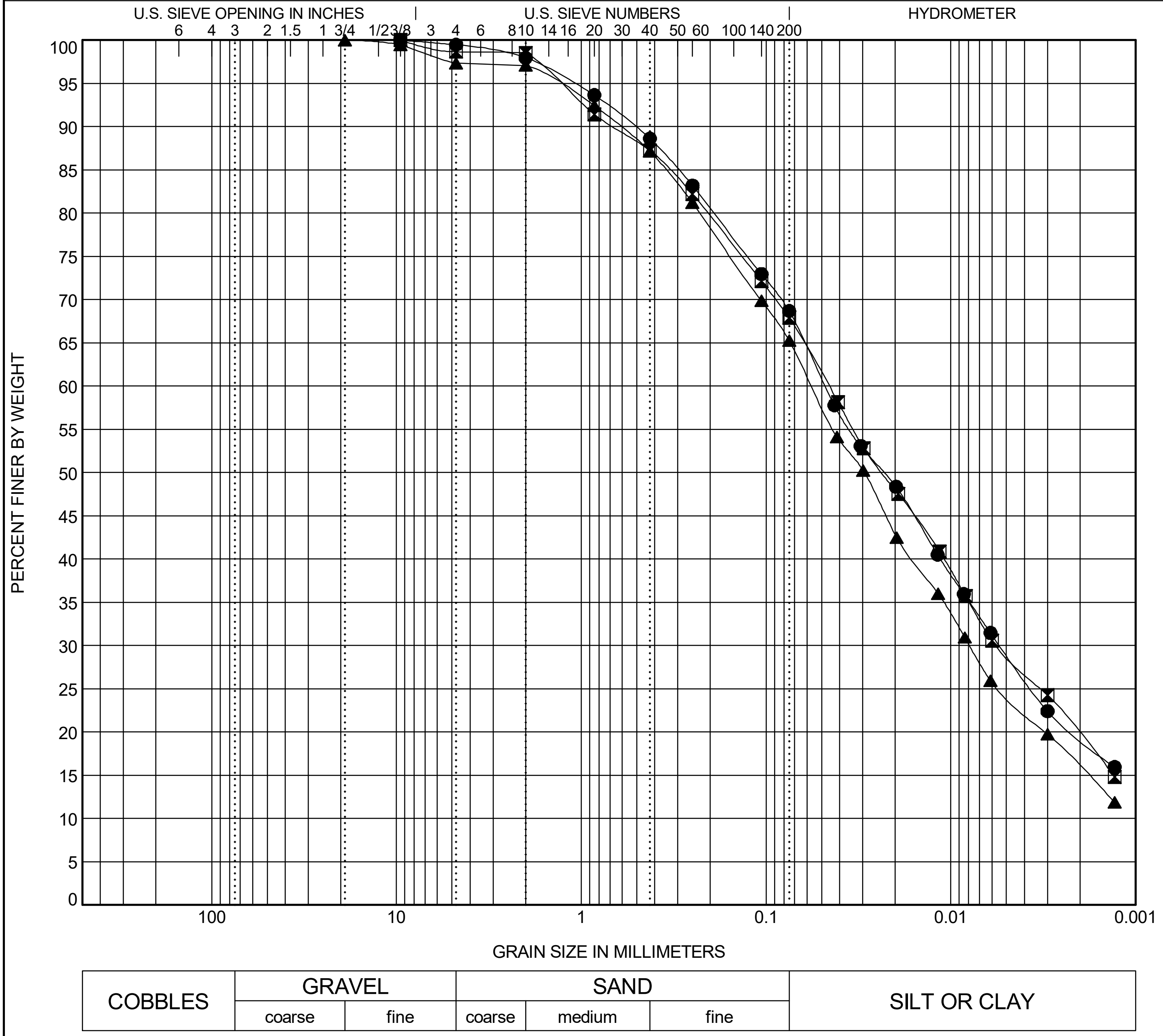
AASHTO R18

CLIENT Detroit Regional Partnership

PROJECT NAME DRP RACER Genesee Industrial Land

PROJECT NUMBER 401.2401267.000

PROJECT LOCATION Genesee Township, Michigan



Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	**SB-01 / SS-4**	**8.5**	**SANDY LEAN CLAY (CL)**	**25**	**13**	**12**		
☒	**SB-02 / SS-7**	**23.5**	**SANDY LEAN CLAY (CL)**	**21**	**12**	**9**		
▲	**SB-04 / SS-3**	**6.0**	**SANDY LEAN CLAY (CL)**	**23**	**13**	**10**		

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**SB-01 / SS-4**	**8.5**	**9.525**	**0.048**	**0.005**		**0.5**	**30.8**	**49.4**	**19.3**
☒	**SB-02 / SS-7**	**23.5**	**9.525**	**0.046**	**0.006**		**1.3**	**30.8**	**48.2**	**19.7**
▲	**SB-04 / SS-3**	**6.0**	**19**	**0.057**	**0.008**		**2.6**	**32.1**	**49.3**	**15.9**

ATTERBERG LIMITS' RESULTS

AASHTO R18

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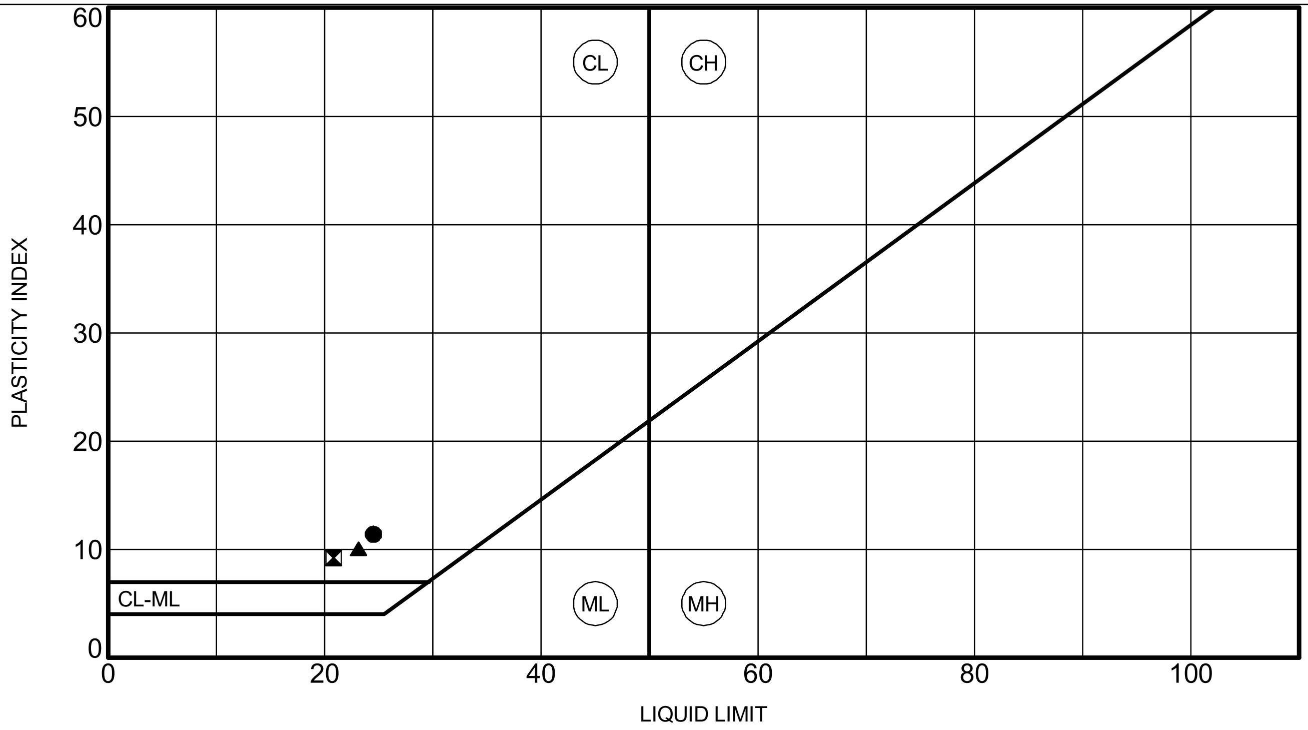

	Specimen Identification		LL	PL	PI	Fines	Classification
●	**SB-01 / SS-4**	**8.5**	**25**	**13**	**12**	**69**	**SANDY LEAN CLAY (CL)**
☒	**SB-02 / SS-7**	**23.5**	**21**	**12**	**9**	**68**	**SANDY LEAN CLAY (CL)**
▲	**SB-04 / SS-3**	**6.0**	**23**	**13**	**10**	**65**	**SANDY LEAN CLAY (CL)**

The Mannik & Smith Group, Inc.
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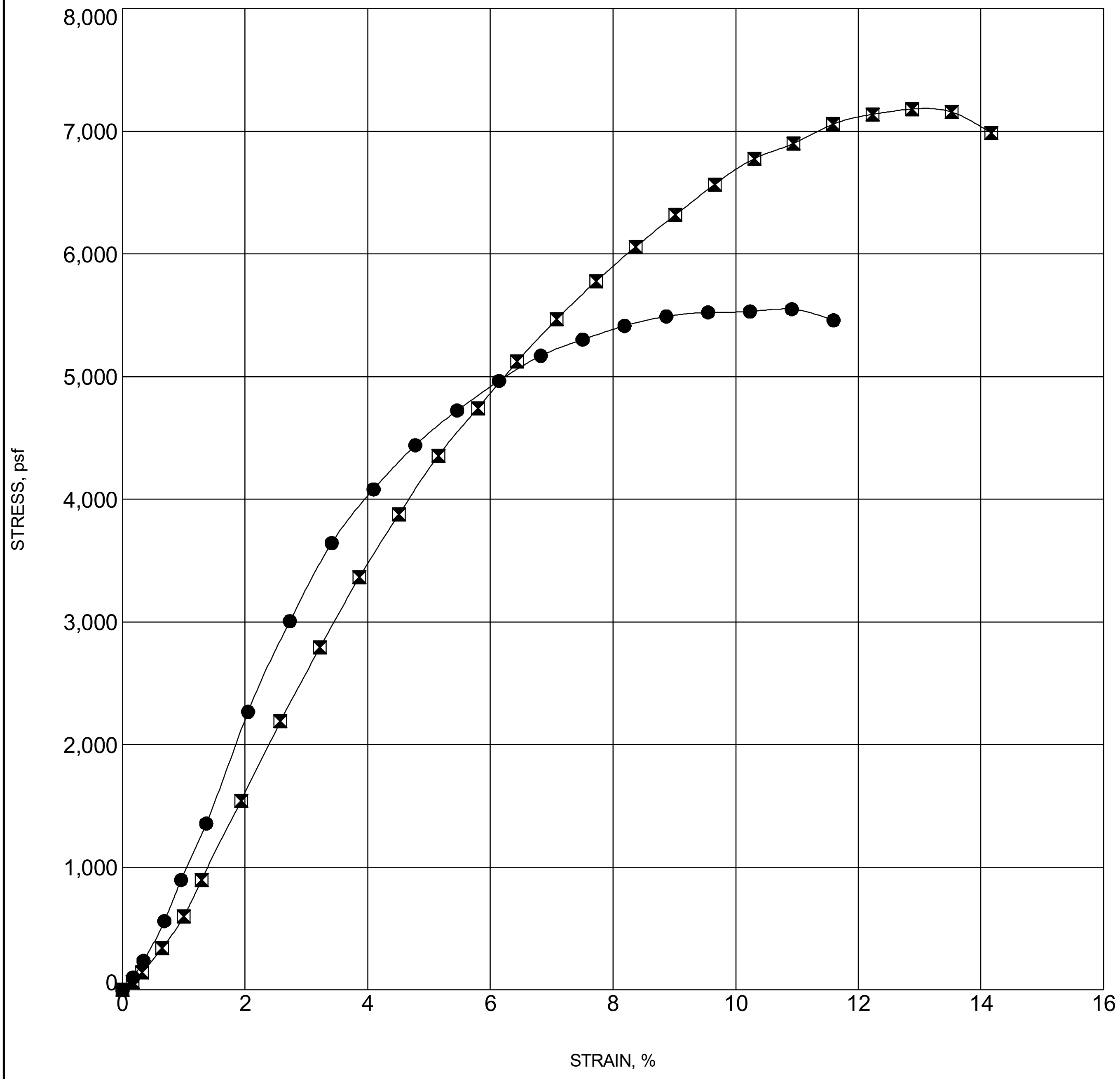
UNCONFINED COMPRESSION TEST

CLIENT Detroit Regional Partnership
PROJECT NAME DRP RACER Genesee Industrial Land
PROJECT NUMBER 401.2401267.000
PROJECT LOCATION Genesee Township, Michigan



Specimen Identification		Classification	UCS (psf)	γ_d	MC%
● SB-03 / SS-3	6.0		5550	117	14
☒ SB-04 / SS-3	6.0	SANDY LEAN CLAY (CL)	7183	125	13