Geotechnical Evaluation Report

Mohawk Valley EDGE
Oneida County, New York

March 2013

O'BRIEN & GERE

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1. INTRODUCTION

1.1 GENERAL

This report has been developed to present geotechnical engineering design recommendations in connection with Mohawk Valley Economic Development Growth Enterprises Corporation (MVEDGE) development of the Marcy Nanocenter at SUNY IT (Marcy Nanocenter) site in Marcy, NY. This geotechnical engineering report provides a summary of the subsurface investigation and recommendations related to the preparation of subsurface soils and foundation design for the proposed structures.

1.2. SITE DESCRIPTION

The site is bounded by Hazard Rd to the north, Morris Rd. to the west, Technology Drive to the south, and Edic Rd. to the East. The site consists of approximately 378 acres, which includes 323 acres owned by the People of the State of New York but is available to MVEDGE for development through a ground sublease and 55 acres of what is referred to as the Farmer Parcel. Approximately 220 acres of the site is utilized for the site development discussed in this report.

1.3. SITE HISTORY

To decrease the overall time-to-market schedule and enhance the marketability of the Marcy Nanocenter site, MVEDGE has progressed site planning and design activities to further understand the baseline site conditions. The information will be used to advance preliminary design elements, which can decrease the overall time an end-user would need to complete design and construction of the facilities. MVEDGE is marketing the site to the semiconductor manufacturing industry. A site layout illustrating a potential full build-out layout (*e.g.*, three fabrication facilities) is provided as Appendix A.

1.4. GEOLOGIC HISTORY

According to the report by Panamerican Consultants, Inc. submitted to O'Brien & Gere in June 2012 titled "Phase IB and II Cultural Resources Investigations for the Proposed Mohawk Valley EDGE Marcy Nanocenter," the Hudson-Mohawk Lowlands is one of four lowland provinces surrounding the Adirondack Mountains (Van Diver 1985:viii, 9). These provinces consist of low, even topography overlying nearly flat Cambrian and Ordovician sedimentation. The Hudson-Mohawk Lowland province is also subdivided into four groups: Mohawk Valley, Hudson Valley, Wallkill Valley, and Shawangunk Mountains. The current project area is located within the Mohawk Valley subdivision, the east-west lowland drained by the Mohawk River, and located between the Adirondacks to the north and the Appalachian Upland to the south. The valley narrows to a deep gorge at Little Falls. Bedrock underlying the study areas is Ordovician shale. This soft shale has been cut through by the Mohawk River to a depth of about 1,000 ft (Cressey 1966:29-31). The soils in the project area are derived from glacial and post-glacial sediments. In the region surrounding the project area, valleys tend to contain narrow, but nearly level alluvial bottom land, where soils normally possess high natural fertility. Other soil characteristics include: good texture, the frequent lack of well-developed structure, and at times poor or excessive drainage problems.

2. PROJECT DESCRIPTION

MVEDGE, the economic development authority for the Mohawk Valley region of Upstate NY, is developing the Marcy Nanocenter site to attract a semiconductor manufacturer and desires the site to accommodate three 450 mm chip fabrication facilities along with the accompanying support structures and utility infrastructure. Proposed nanocenter operations provide synergies with SUNY IT curriculum, the proposed Computer Chip Commercialization Center (Quad C) and on-going nanotechnology activities in the Capital District of the State.

2.1 EACH BUILDING

The Conceptual Site Plan (Appendix A) shows the layout of the chip fabrication facilities along with associated support buildings, utility yards, and access road. The following is a summary of the main features of the Conceptual Site Plan:

- FAB – The FAB, or Fabrication, buildings are all sized to manufacture microelectronics at a 450 mm wafer size. This facility provides a 370,000 sf gross manufacturing clean room floor within a total facility size of 520,000 sf site footprint, including all manufacturing, manufacturing support spaces, electrical distribution, mechanical space, chemical spaces and required life safety area.
- CUB – The CUB, or Central Utility Building, is sized to house all major systems within its 437,000 sf site footprint. Within this footprint all major mechanical, electrical and process chemical support for the entire site will be housed. These systems will include boilers, chillers, cooling towers, UPW and waste treatment, normal and continuous power systems and major electrical distribution systems.
- RD – The RD, or Research and Development, building will house all of the back end processes for the facility. This 200,000 sf gross manufacturing facility will house the processes that are typically outside of the normal "FAB" structure, but are needed on site in a "Foundry" concept. These spaces include, but are not limited to Lot Start and Lot Ship, Test, Polyimide and Material testing labs.
- Office – The center of a facility of this type is the administration building. Built to house not only the staff on the site, but also the front of house presentation spaces. To house all of these requirements, we have planned for an 8 story, 250,000 sf gross floor plate building, providing the area required for the entire facilities staff.
- HPM – The HPM warehouse, is the location where all liquid hazardous chemicals are stored in bulk storage for use in the fabrication process. This facility is planned with expandability to allow for staged construction.
- Gas Yard – The Gas Yard it where all bulk gases are distributed from. This area will have not only large tanks, but also a significant distribution network of piping and trestles leading to each FAB, the RD building and the CUB.

2.2 ROADWAYS, DOCKS, AND RETAINING FEATURES

In addition to the above structures, there are also access roads, parking areas, loading docks, and retaining walls proposed for the site. General recommendations for each of these features will be made based on results of the subsurface investigation.

3. SUBSURFACE INVESTIGATION

3.1 BORING LOGS

Geotechnical soil borings were drilled by NYEG Drilling LLC. at locations selected by The M+W Group and indicated on Figure 1. An O'Brien & Gere representative was present on-site to observe the drilling activities. The borings were drilled using both tire and track mounted off-road drill rigs.

A total of 72 borings were drilled as part of the subsurface investigation program (OBG-7 to OBG-78). Of the72 borings drilled, 4 borings (OBG-28, OBG- 36, OBG-43, and OBG-53) included rock cores. A summary of the soil borings is presented in Appendix 2. The soil boring logs are present in Exhibit 1.

The soil borings were drilled using 3¼-inch inside diameter hollow stem augers in accordance with ASTM D1452, "Standard Practice for Soil Investigation and Sampling by Auger Borings." The soil borings were drilled to depths of approximately 28.2 to 48.2 ft below grade. The soil borings were sampled continuously from the surface to the bottom of each boring. Split spoon sampling was conducted in accordance with ASTM D1586, "Standard Method for Penetration Test and Split Barrel Sampling of Soils." The rock cores were collected using a NQ core barrel in accordance with ASTM D2113, "Standard Practice for Rock Core Drilling and Sampling of Rock for Site Investigation." A summary of the rock cores are present in Appendix 2.

3.2 GROUNDWATER MONITORING

Piezometers or monitoring wells were installed as part of the investigation. Groundwater elevations were noted during drilling through observation of the samples and also by measuring groundwater depth in each borehole following completion of drilling.

3.3 SHEAR WAVE VELOCITY

No shear wave velocity analysis was performed as part of this investigation. It is recommended that the structural engineer designing the buildings obtain the appropriate shear wave velocity values from the 2010 Building Code of New York State

3.4 SOIL PERMEABILITY AND PERCOLATION

Based on the subsurface investigation, the overburden soils primarily consist of fine grained soils with some gravel and sands present. According to the Unified Soil Classification System and the grain size analysis results, the overburden soils would generally be classified as a "SC" soil. Based on the Naval Facilities Engineering Command (NAVFAC) Design Manual 7.02, page 7.2-39 Table 1 "Typical Properties of Compacted Soils," the permeability of the overburden soils would be approximately $1x10^{-5}$ ft/min.

4. SUBSURFACE CONDITIONS

4.1 OVERBURDEN DESCRIPTION

The overburden at the site generally consist of less than 6 inches of topsoil overlying brown to gray, loose to dense sand with varying amounts of clay, silt and gravel. This overburden layer generally transitions to a residual shale layer consisting primarily of silt and fragments of weathered shale. The thickness of this layer generally increases the further south you move across the site and in small sections along the west side and northeast corner. Figures 2-5 show soil profiles along select sections across the site.

4.2 BEDROCK

The bedrock at the site generally consisted of black, highly to moderately weathered Utica Shale. The depth to the top of bedrock generally increased along the site from the north to the south end. The rock profiles are shown in Figures 2-5. Based on the ease of augering the weathered shale during drilling, it appears that removal can be achieved with conventional excavation equipment.

4.3 GROUNDWATER

Groundwater was observed in 45 of the 72 borings during drilling at depths ranging from grade to approximately 23 feet below grade. Based on review of the boring logs, groundwater appears to be perched and a distinct water table was not observed.

4.4 SEISMIC SITE CLASS

Review of the boring logs indicates that the soils at the Marcy Nanocenter site classify as Site Class D according to Table 1615.1.1 of the Building Code of New York State. This Site Class designation should be used when determining the maximum considered earthquake spectral response accelerations as required in the Building Code of New York State.

5. LABORATORY TESTING

Following review of the boring logs and the proposed building elevations, a laboratory testing program was developed. The testing program included the selection of several borings along each row of FAB structures. To mimic anticipated construction practices of utilizing stockpiled weathered shale as fill under these structures, and to account for the limited volume of material collected, samples were composited for analysis. Two composited samples were analyzed and were comprised of the samples detailed below in Table 1.

Table 1. Laboratory Composite Sample Summary

Boring	Samples
Composite 1	
OBG-26	S-3 through S-23
OBG-27	S-7 through S-23
OBG-28	S-7 through S-13
OBG-29	S-5 through S-18
OBG-30	S-6 through S-18
OBG-33	S-5 through S-15
OBG-34	S-4 through S-15
OBG-35	S-5 through S-15
OBG-36	S-5 through S-11
OBG-37	S-7 through S-14
Composite 2	
OBG-52	S-21 through S-23
OBG-53	S-9 through S-12 / S-15 through S-21
OBG-54	S-16 through S-23
OBG-55	S-6 through S-22
OBG-56	S-11 through S-25
OBG-57	S-15, S-16, S-18, S-19, S-22, and S-23
OBG-59	S-4 through S-15
OBG-60	S-4b through S-17
OBG-61	S-14 through S-19
OBG-67	S-10 through S-16
OBG-68	S-14 through S-17
OBG-69	S-19 through S-23
OBG-71	S-8 through S-15

Source: O'Brien & Gere

Laboratory tests for each sample included the following:

- grain size
- Atterberg limits
- slake test
- swell test
- consolidation test
- modified proctor test
- chlorides

- sulfites
- pH
- electrical resistivity

A summary of the results for each analysis are discussed below.

5.1 GRAIN SIZE

Grain size analysis was performed on the composite samples in accordance with ASTM D422-63 "Standard Test Method for Particle-Size Analysis of Soils." Results of the sieve analysis indicate the weathered shale if excavated would consist of primarily of clayey sand with gravel and are classified as a SM soil according to the USCS Soil Classification System. Each of the composite samples consisted of approximately 20% fines. Results of the grain size analysis are presented in Exhibit 2.

5.2 ATTERBERG LIMITS

Atterberg Limits were performed on the minus No. 40 sieve material collected during the grain size analysis of the composite samples in accordance with ASTM D4318-10 "Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils." The results indicate that the fine grained soils can be classified as a CL-ML soil in accordance with the USCS. Detailed laboratory results are presented in Exhibit 2.

5.3 SLAKE TEST

A Slake Durability Test was conducted the composite samples in accordance with ASTM D4644-04 "Standard Test Method for Slake Durability of Shales and Similar Weak Rocks." Both samples were subjected to two cycles. Analysis after the second cycle resulted in a Slake Durability Index for Composites 1 and 2 of 89.0 and 95.0 %, respectively. The remaining fragments for both Composites 1 and 2 were classified as type 2 as the retained material consisted of large and small pieces. Review of photos taken during the tests also indicate that slaking of the residual shale is not anticipated to be a concern. Detailed laboratory results are presented in Exhibit 2.

5.4 SWELL TEST

A One-Dimensional Swell Test was conducted on the composite samples in accordance with Method C of ASTM D4546-96 "Standard Test Method for One-Dimensional Swell or Settlement Potential of Cohesive Soils." Results indicate that the weathered shale is not anticipated to exhibit any swelling tendencies. Detailed laboratory results are presented in Exhibit 2.

5.5 CONSOLIDATION TEST

A One-Dimensional Consolidation Test was conducted on the composite samples in accordance with ASTM D2435 "Standard Test Methods for One-Dimensional Consolidation Properties of Soils Using Incremental Loading." Analysis shows that the shale samples from Composites 1 and 2 showed maximum deflections under 16 tsf of approximately 0.35 and 0.3 inches, respectively. Detailed laboratory results are present in Exhibit 2.

5.6 MODIFIED PROCTOR TEST

A modified proctor test was performed on the composite samples in accordance with ASTM D4718 "Standard Practice for Correction of Unit Weight and Water Content for Soils Containing Oversize Particles and ASTM D1557 Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft lbf/ft^3 (2,700 kN m/m^3))." Results for Composite 1 indicate a Corrected optimum water content and maximum dry density of 5.9% and 138.8 pcf, respectively. The analysis of Composite 2 resulted in a corrected optimum water content and maximum dry density of 5.9% and 139.4 pcf, respectively. Detailed laboratory results are presented in Exhibit 2.

5.7 CHLORIDES

The chloride content present in the composite samples were analyzed in accordance with AASHTO T 291 "Water Soluble Chloride Content in Soil (Method B Extraction)." Tests were conducted with the weathered shale fragments captured with the No. 10 sieve and above. Results show chloride concentrations in the rock fragments from Composites 1 and 2 of 6.5 and 6.94 mg/L, respectively. Detailed laboratory results are presented in Exhibit 2.

5.8 SULFATES

The sulfate ion content present in the composite samples was analyzed in accordance with AASHTO T 290-95 "Standard Test Method for Determining Water-Soluble Sulfate Ion Content in Soils." Results indicate that the sulfate ion concentrations in Composites 1 and 2 are 210.44 and 318.88 mg/L (ppm), respectively. The sulfate ion content when corrected for moisture in Composites 1 and 2 was found to be 636.7 and 966.1 mg/kg, respectively. Detailed laboratory results are presented in Exhibit 2.

5.9 PH

A pH analysis was conducted on the composite samples in accordance with ASTM D4972-01 "Standard Test Method for pH of Soils." Two tests were conducted on each sample and results indicate a pH of Composites 1 and 2 of 7.4 and 7.6, respectively. Detailed laboratory results are provided in Exhibit 2.

5.10 ELECTRICAL RESISTIVITY

The electrical resistivity of the composite samples was analyzed in accordance with AASHTO T288-12 Standard Test Method for Determining Minimum Laboratory Soil Resistivity. The results indicate that resistivity varies from 1,150 to 13,000 ohm-cm for the soil sampled dependent on moisture content of the soil. Based on the observed moisture content of the residual shale layer, it is believed that the electrical resistivity should be greater than 2000 ohm-cm. Detailed laboratory test results are presented in Exhibit 2.

6. SOIL CHARACTERISTICS

6.1 EXCAVATION AND SLOPE STABILITY

Based on the subsurface conditions observed, the excavations required for the construction of foundations and retaining structures may generally be performed by conventional open cut methods using standard construction techniques and equipment. Excavations should be performed in accordance with all applicable Occupational, Safety, and Health Act requirements. Earthwork should commence with the complete removal of all topsoil, fill, and native soils as required to attain proposed finished subgrade elevations. Areas which are unsuitable should be over-excavated to a suitable bearing stratum, and replaced with properly compacted structural fill or native soils, as recommended or directed.

Open cut methods should be based on recommended maximum slopes of 1.5H:1V. If the excavation size must be limited, an excavation support system that protects adjacent roads or structures should be designed by a New York State licensed engineer and submitted for review. Existing building footings should be protected by underpinning and or sheeting and shoring system installed prior to excavation of new building footings or mat foundation system. Conventional open cut methods and/or trench boxes may be used for construction and installation of utilities.

6.2 SUITABILITY OF ON-SITE SOILS AS FILL MATERIAL

It is anticipated that the on-site native soils may be used as backfill at the site where possible to minimize the quantity of imported fill required to establish the proposed grades. It is recommended that overburden soils may be reused as non-structural backfill against the outside of exterior walls, provided they do not contain substantial amounts of organics (*i.e.* topsoil). It is not recommended that the overburden soils to be used as backfill under roads or parking lots. It is not recommended that the residual shale material not be used as structural fill as it contains a large percentage of fine-grained material that may be difficult to place and compact properly.

Based on the results of the laboratory analysis, it is anticipated that the weathered shale bedrock may be difficult to be used as structural backfill. Reuse of the on-site materials will largely be contingent upon moisture control of the stockpiled material. Based on results of the laboratory analysis, it appears that the weathered shale is comprised largely of silt. If construction is performed during wet seasons (*i.e.* late fall, winter, early spring), it may be difficult to attain proper compaction, in which case an off-site granular structural fill should be used.

6.3 FILL COMPACTION

On-site soils placed as fill within or adjacent to the structural footprints should be placed in horizontal lifts not to exceed 10 inches of loose lift thickness and compacted to a minimum of 95% of the maximum dry density as determined by ASTM Method D1557 (Modified Proctor Test). Imported fill, backfill, and base course materials beneath foundations and floor slabs should be placed in horizontal lifts not to exceed 10 inches loose thickness, and should be compacted to 95% of maximum dry density according to the Modified Proctor Test. The results of the Modified Proctor Test performed on the weathered shale are presented in Exhibit 2.

In areas where general fill is required to establish grades, but not in areas of structural fill, soils should be placed in horizontal lifts not to exceed 10 inches of loose lift thickness. These areas should be compacted to a minimum of 90% of the maximum dry density as determined by ASTM Method D1557.

6.4 EROSION POTENTIAL

Review of the subsurface conditions at the site indicate that there is a high potential for erosion to occur. This is primarily due to the fine grained nature of the overburden and residual shale layers. The poor drainage qualities of the site soils was also observed during the subsurface investigation. Stockpiled soil should also be vegetated to minimize erosion.

6.5 LIQUEFACTION POTENTIAL

Review of the subsurface conditions at the site indicates that the potential for liquefaction of the soils at the site is unlikely to occur. The relative density and composition of the soils observed in the borings are not consistent with soils likely to liquefy.

6.6 CORROSION POTENTIAL

The corrosion potential of the soils was analyzed based on Table A.1 “Soil-test Evaluation” of ANSI/AWWA C105/A21.5-10. Based on the laboratory results and the ten-point system, the soils do not appear to be corrosive.

6.7 ELECTRICAL RESISTIVITY

The electrical resistivity was analyzed in accordance with AASHTO T288-12 Standard Test Method for Determining Minimum Laboratory Soil Resistivity. Based on the results and field observations, the resistivity varies from 2,000 to 13,000 ohm-cm for the soil sampled dependent on moisture content of the soil. Detailed laboratory test results are presented in Exhibit 2.

6.8 COMPRESSIBILITY AND SWELL

Compressibility and swell was analyzed as part of the laboratory testing program. Results indicate that the soils would not compress or swell under the expected loadings. Detailed lab results are present in Exhibit 2.

7. SITE PREPARATION RECOMMENDATIONS

7.1 SITE CLEARING

The Marcy Nanocenter site is currently heavily vegetated with a variety of grass, brush and trees. Based on the proposed site layout and finished elevations, an extensive amount of clearing and grading will be required to prepare the site for construction. It is recommended that all vegetation, topsoil, and overburden soils containing organics should be cleared and removed from the proposed project location. Saturated overburden soils shall also be removed and allowed to dewater if they are suitable to be used as general backfill across the site. Topsoil may be stockpiled for use in grassed areas but should be located away from areas of excavation.

It is recommended that once the clearing of the site is conducted, the exposed native soils should immediately be seeded and mulched in accordance with the Stormwater Pollution Prevention Plan (SWPPP) report submitted by O'Brien & Gere in December, 2012. To further minimize erosion, it is recommended that the areas beyond what is needed for construction remain vegetated wherever possible.

7.2 SUBGRADE PREPARATION

Subgrade preparation should consist of the removal of all vegetation and soil above the weathered shale layer to the proposed elevations of the foundations. The exposed undisturbed shale should be proof rolled a minimum of six passes with a roller having a minimum weight of 10 tons. If small excavations are made where use of a roller is prohibited, sub-grade soils should be compacted with equipment appropriate for the size of the excavation (*e.g.* vibratory plate compactor mounted on excavator). If any soft spots are found, the area should be marked out, excavated and backfilled with compacted structural fill in accordance with the Bulk Fill Placement and Compaction section of this report. The foundations should be placed as soon as possible after excavation and backfilled as soon as possible after the concrete has been placed and cured.

If the weathered shale layer is exposed to precipitation, it may be difficult to achieve proper compaction. If this occurs, it is recommended that the weathered shale layer be allowed to dewater or be removed and replaced with a structural fill.

7.3 BULK FILL PLACEMENT AND COMPACTION

Imported material placed as fill within the building footprints should be structural fill consisting of predominantly granular soils, free from organic matter, ice, debris, or other deleterious material. The structural fill should be in accordance with the technical specifications

On-site soils placed as fill within or adjacent to the structural footprints should be placed in horizontal lifts not to exceed 10 inches of loose lift thickness and compacted to a minimum of 95% of the maximum dry density as determined by ASTM Method D1557 (Modified Proctor Test). Imported fill, backfill, and base course materials beneath foundations and floor slabs should be placed in horizontal lifts not to exceed 10 inches loose thickness, and should be compacted to 95% of maximum dry density according to the Modified Proctor Test.

7.4 SETTLEMENT MONITORING

Based on the site topography and proposed finished floor elevations, it is anticipated that some areas will require large amounts of structural fill. Once the fill is placed in accordance with the bulk fill placement section of this report, it is recommended that surveying points be taken and monitored for six months to allow for any secondary consolidation to occur. It is also recommended that a series of plate load test be performed across the site in areas where backfilling has occurred. The plate load tests should be conducted in critical areas of construction such as buildings or retaining features in accordance with ASTM Standard D1196 "Standard Test Method for Nonrepetitive Static Plate Load Test of Soils and Flexible Pavement Components, for Use in Evaluation and Design of Airport and Highway Pavements." It is recommended that O'Brien & Gere be contacted to assist in selecting locations for plate load tests and to oversee the testing.

8. SHALLOW FOUNDATION RECOMMENDATIONS

Shallow foundations are proposed for the structures at the site. The shallow foundations will consist of a mat and spread type foundations. To properly design shallow foundations, bearing capacity and anticipated settlement must be considered.

8.1 DESIGN BEARING CAPACITY

The bearing capacity analysis was conducted using the Terzaghi bearing capacity equation with correlated internal friction values. The bearing capacity calculations assume that the spread footings and mat foundations will be placed on top of undisturbed native shale or structural select fill and be a minimum of 4 ft below finished grade for frost protection. Based on preliminary building descriptions and expected foundation sizes, a bearing capacity of 4.0 kips per square foot (ksf) can be safely supported by the native shale.

The calculated bearing capacities have also been verified using presumptive bearing capacity correlations, which are evaluated considering the type of structural fill to be placed and the type of soil conditions anticipated to be encountered. The calculated soil bearing capacities also correlate well with the presumptive bearing capacity which considers the soil type and the average standard penetration (N) value of the supporting soil layer.

8.2 ESTIMATED SETTLEMENT

8.2.1 Mat Type Foundations

Mat type foundations are proposed for use on the central utility building (CUB) and on each of the three fabrication buildings (FAB) shown on the proposed site layout present in Appendix A. The CUB was analyzed as a single mat of size 1250 ft x 350 ft with a uniform loading of 4.0 ksf sitting on 15 ft of structural fill overlaying weathered bedrock. Results of the settlement analysis calculate an estimated total settlement of approximately 0.5 inches and a differential settlement of less than 0.25 inches. Each of the three FAB's were analyzed as a single mat of size 900 ft x 600 ft sitting on 30 ft of structural fill. The estimated total settlements of the mat foundations are calculated to be approximately 1.0 inch based on a loading of 4.0 ksf. Based on the topography of the site and the amount of structural fill that is expected to be placed, the total differential settlement will be approximately 0.5 inches across each building.

8.2.2 Spread Type Foundations

Spread type foundations are recommended for use on the proposed office building, research and development building, and the HPM warehouse. Settlement calculations for each building were performed under the assumption that the foundations would consist of 6ft x 6ft spread footings and 3 ft wide strip footings sitting on 30 ft of structural fill overlaying weathered bedrock. The estimated total settlements of office building, research and development building, and the HPM warehouse are projected to be less than 0.25 inch based on a loading of 4 ksf. Based on the proposed finished floor elevations and the amount of structural fill expected to be placed, the total differential settlement will be less than 0.25 inch.

8.3 DESIGN SUBGRADE MODULUS

Design of mat type foundations and floor slabs should be based upon a modulus of subgrade reaction of 350 kips per cubic foot (200 pounds per cubic inch) if placed directly on compacted weathered shale or a select structural fill. Floor slabs should bear upon native shale or select structural fill in accordance with the subgrade preparation and bulk fill placement and compactions criteria sections of this report and the technical specifications. It is recommended that these values be confirmed through a series of plate load tests.

9. RETAINING WALL RECOMMENDATIONS

9.1 BASEMENT AND LOADING DOCK WALLS

It is anticipated, due to proposed elevations of the buildings and finished grade, that walls for the buildings and loading docks will be constructed below grade. The below grade walls will be subjected to either active or at-rest lateral earth pressure, depending upon the degree of restriction to movement of the walls during the time of construction. For walls which are permitted to rotate or translate at the top, this represents an active condition with an active earth pressure. However, for rigid walls with movement restricted, this represents an at-rest condition. Since groundwater was encountered at the time of drilling, walls should be designed to withstand hydrostatic pressure. The recommended lateral earth pressure coefficients for the native soils are provided in Table 2. The lateral earth pressures were developed based on Naval Facilities Engineering Command (NAVFAC), Design Manual 7.02, pages 7.2-39, 63 and 64.

Table 2 ***Subgrade Structures Wall Design Parameters***

Parameter	Value
Static coefficient of sliding friction	
Between concrete and on-site weathered shale backfill	0.35
Between concrete and imported granular structural fill	0.45
Unit weight	
On-site natural soils (Weathered Shale)	120 pcf
Imported granular structural fill	125 pcf
On-site natural soils, ϕ =32°	
Coefficient of active earth pressures – Ka	0.31
Coefficient of passive earth pressures – Kp	3.25
Coefficient of at-rest earth pressures – Ko	0.47
Imported granular structural fill, ϕ =36°	
Coefficient of active earth pressures – Ka	0.225
Coefficient of passive earth pressures – Kp	3.8
Coefficient of at-rest earth pressures – Ko	0.41

9.2 SITE RETAINING WALLS

To establish the proposed foundation elevations, it is anticipated that retaining walls will be required. Based on the expected wall heights and the subsurface conditions observed, it is recommended that retaining walls be constructed of mechanically stabilized earth (MSE), cantilevered wall or of a secant wall system. The MSE walls should consist of granular soils reinforced with geogrids installed at distinct elevations of the wall. The cantilevered retaining wall system would be constructed using steel soldier piles installed in holes bored in bedrock and backfilled with cast-in-place concrete. Once installed, pre-cast concrete or wooden lagging will be placed between the steel solider beams. Secant pile walls consist of a series of reinforced concrete drilled piles installed in an overlapping pattern to form a uniform wall down to bedrock.

No design of site retaining walls has been included in this report, but it is anticipated that it will be performed following development of the site layout and building locations. Design of the site retaining walls should be designed based on the parameters listed above in Table 2.

10. PAVEMENT DESIGN AND CONSTRUCTION RECOMMENDATIONS

It is recommended that all roadways and parking areas should be constructed on select structural fill. All topsoil, soils containing organics and saturated soils should be removed in areas where roadways and parking areas are to be constructed. It is recommended that the subgrade be compacted in accordance with section 7.2, Subgrade Preparation, of this report. Based on the subsurface conditions observed and Table 1 "Typical Properties of Compacted Soils" of the NAVFAC design manual 7.02, it is recommended that pavement designs be based on a California Bearing Ratio of approximately 10 assuming that the subgrade preparation has been performed in accordance with this report. It is also recommended that the use of a triaxial geogrid be utilized in the design of the pavement to minimize the amount of fill required and to increase the lifespan of the pavement section.

11. SPECIAL INSPECTIONS

Through the duration of construction, it is recommended that a geotechnical engineer from O'Brien & Gere be on-site for the inspection and confirmation that all proof rolling, subgrade preparations, plate load tests, and foundation construction be conducted in accordance with the specifications outlined in this report.

12. CLOSURE

This geotechnical engineering report has been prepared by O'Brien & Gere Engineers, Inc. for the exclusive use of Mohawk Valley EDGE and O'Brien & Gere Engineers, Inc. for the Marcy Nanocenter development. The recommendations in this report are based on the information obtained from the subsurface investigation and our understanding of the proposed construction. Changes to the recommendations may be warranted if the actual subsurface conditions vary from those anticipated, or if the proposed structure varies from that discussed in this report. In addition, construction operations at or adjacent to the Site and natural events such as floods, earthquakes, or ground water fluctuations may also affect subsurface conditions.

REFERENCES

Cressey, George B. (1966). Land Forms. In *Geography of New York State*, edited by John H. Thompson, pp. 19-53. Syracuse University Press, Syracuse, NY.

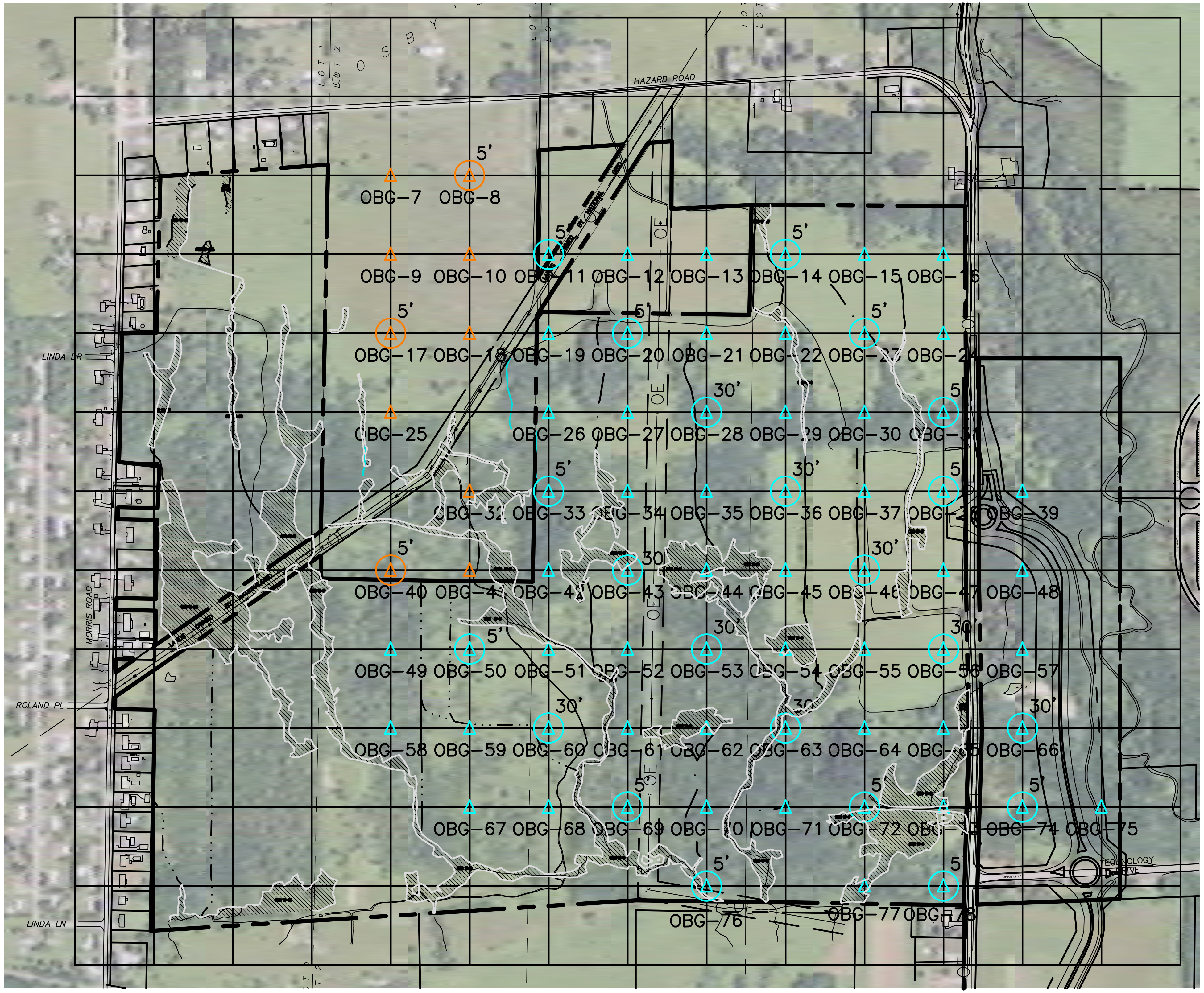
Naval Facilities Engineering Command, (1986). *Design Manual 7.02 – Foundations & Earth Structures*

Panamerican Consultants, Inc. (June 2012). *Phase IB and II Cultural Resources Investigation for the Proposed Mohawk Valley Edge Marcy Nanocenter*.

Van Diver, Bradford B, (1985). *Roadside Geology of New York.* Mountain Press Publishing Company, Missoula, MT.

Boring Location Map

I:\Mohawk-Vly-Edge.10316\48992.Marcy-Nanocente\Docs\DWG\Sheets\BORING PLAN.dwg Jul 19, 2012 - 8:48am



LEGEND

OBG-2 PROPOSED BORING LOCATION FARMER PARCEL

OBG-2 PROPOSED BORING LOCATION MARCY NANOCENTER PARCEL

X' OBG-3 X' OBG-3 ROCK CORE AND DEPTH OF CORE

WETLANDS

1"=600' 600 0 600

PRELIMINARY NOT FOR CONSTRUCTION

DATE: 07/19/12

IT IS A VIOLATION OF LAW FOR ANY PERSON, UNLESS ACTING UNDER THE DIRECTION OF A LICENSED ENGINEER, TO ALTER THIS DOCUMENT.

IN CHARGE OF ____

DESIGNED BY ____ CHECKED BY ____

DRAWN BY ____

NO.	DATE	REVISION	INIT.

O'BRIEN & GERE

MOHAWK VALLEY EDGE
MARCY NANOCENTER
RING ROAD DEVELOPMENT
MARCY, NEW YORK
BORING PLAN

FILE NO. 10316.48992-002

DATE JUNE 2012

Soil Profile Cross Section

Cross-Section A-A'

Soil Profile Cross Section

OBG-25 through OBG-31 (B - B')

Soil Profile Cross Section

OBG-40 through OBG-48 (C - C')

Soil Profile Cross Section

OBG-49 through OBG-57 (D - D')

Conceptual Site Plan

IT IS A VIOLATION OF LAW FOR ANY PERSON, UNLESS ACTING UNDER THE DIRECTION OF A LICENSED ENGINEER, TO ALTER THIS DOCUMENT.

THIS DRAWING WAS PREPARED AT THE SCALE INDICATED IN THE TITLE BLOCK. INACCURACIES IN THE STATED SCALE MAY BE INTRODUCED WHEN DRAWINGS ARE REPRODUCED BY ANY MEANS. USE THE GRAPHIC SCALE BAR IN THE TITLE BLOCK TO DETERMINE THE ACTUAL SCALE OF THIS DRAWING.

IN CHARGE OF ________
DESIGNED BY ______ CHECKED BY ______
DRAWN BY ________

1"=200' 200 0 200 400

NO.	DATE	REVISION	INIT.

O'BRIEN & GERE

MOHAWK VALLEY EDGE
MARCY NANOCENTER
MARCY, NEW YORK

CIVIL
GENERAL PLAN

FILE NO. 10316.48992-GEN1
DATE JANUARY 2013

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Soil Boring and Rock Core Summaries

Boring Log Summary Table

OBG Boring Number	Existing Elevation	Boring Location	Depth of Boring	Water Table			Rock Core	
				While Drilling	Before Casing Removed	After Casing Removed	Depth to Top of Weathered Rock	Elev of Top of Rock
OBG-7	657.0	OL	38.0	NN	NN	NN	8.0	649.0
OBG-8	664.7	OL	28.2	NN	NN	NN	4.5	660.2
OBG-9	640.1	OL	30.0	NN	NN	NN	22.0	618.1
OBG-10	647.4	OL	28.2	NN	18.5	25.5	16.0	631.4
OBG-11	647.6	OL	28.2	NN	18.5	25.5	6.5	641.1
OBG-12	642.5	OL	28.3	NN	NN	NN	6.1	636.4
OBG-13	635.2	OL	28.3	8.8	24.0	8.0	14.0	621.2
OBG-14	628.8	OL	28.3	11.0	9.5	13.2	13.5	615.3
OBG-15	623.7	OL	28.3	2.4	10.0	13.2	NN	-
OBG-16	616.3	OL	28.2	NN	NN	NN	20.0	596.3
OBG-17	623.2	OL	28.0	NN	NN	NN	27.0	596.2
OBG-18	625.0	OL	28.2	19.4	24.9	NN	9.5	615.5
OBG-19	627.6	OL	28.2	6.8	10.0	NN	10.0	617.6
OBG-20	624.1	OL	28.1	NN	NN	NN	4.0	620.1
OBG-21	619.2	OL	28.3	19.3	NN	NN	24.0	595.2
OBG-22	607.1	OL	28.2	6.0	25.0	13.4	10.0	597.1
OBG-23	603.8	OL	28.1	NN	NN	NN	5.5	598.3
OBG-24	601.8	OL	28.2	9.9	14.5	NN	11.5	590.3
OBG-25	594.3	OL	30.0	6.8	10.0	NN	NN	-
OBG-26	592.2	OL	44.2	8.6	25.0	NN	4.0	588.2
OBG-27	594.7	OL	44.1	NN	NN	NN	12.0	582.7
OBG-28	592.4	GPS Location	24.8	NN	-	6.5	10.0	582.4
OBG-29	586.0	GPS Location	34.1	14.5	14.5	NN	8.0	578.0
OBG-30	582.4	OL	34.2	20.0	16.0	15.2	10.0	572.4
OBG-31	575.8	OL	34.1	NN	NN	NN	32.0	543.8
OBG-32	554.4	OL	28.2	2.0	NN	9.0	5.8	548.6
OBG-33	557.5	5ft 225deg SW	28.1	NN	NN	NN	7.2	550.3
OBG-34	555.9	GPS Location	28.3	NN	NN	NN	6.0	549.9
OBG-35	554.7	GPS Location	28.1	NN	NN	16.0	8.0	546.7
OBG-36	553.2	OL	19.7	NN	-	6.0	8.0	545.2

Boring Log Summary Table

OBG Boring Number	Existing Elevation	Boring Location	Depth of Boring	Water Table			Rock Core	
				While Drilling	Before Casing Removed	After Casing Removed	Depth to Top of Weathered Rock	Elev of Top of Rock
OBG-37	548.7	OL	34.5	7.0	12.0	12.3	12.0	536.7
OBG-38	545.3	OL	32.2	NN	NN	NN	10.0	535.3
OBG-39	542.2	OL	28.2	8.0	25.0	12.0	7.5	534.7
OBG-40	539.0	GPS Location	28.3	22.0	16.1	10.0	12.2	526.8
OBG-41	534.9	GPS Location	28.2	4.8	17.9	10.0	10.4	524.5
OBG-42	534.2	GPS Location	28.1	7.0	7.0	2.0	10.9	523.3
OBG-43	533.6	GPS Location	29.0	5.5	-	2.0	9.0	524.6
OBG-44	530.5	GPS Location	28.2	5.5	NN	15.0	6.0	524.5
OBG-45	530.3	OL	28.2	22.0	27.9	14.9	11.0	519.3
OBG-46	526.8	OL	28.1	0.3	16.0	8.0	8.0	518.8
OBG-47	528.0	OL	28.2	2.0	27.3	2.0	12.0	516.0
OBG-48	526.1	OL	28.1	7.0	16.0	12.0	24.1	502.0
OBG-49	524.2	39.5ft 91deg E	28.1	4.0	NN	4.3	13.0	511.2
OBG-50	525.2	OL	36.2	4.0	NN	9.0	14.0	511.2
OBG-51	522.9	OL	48.1	4.0	40.0	NN	17.0	505.9
OBG-52	515.8	GPS Location	46.2	3.5	NN	15.0	24.0	491.8
OBG-53	514.0	GPS Location	40.2	11.0	-	9.0	15.4	498.6
OBG-54	514.6	GPS Location	44.1	10.0	10.0	10.0	30.0	484.6
OBG-55	514.1	21ft 150 deg SE	42.2	3.0	3.0	NN	10.0	504.1
OBG-56	518.9	7ft 90 deg E	48.2	6.6	NN	NN	19.8	499.1
OBG-57	520.6	14ft E	48.0	23.0	15.0	17.0	42.0	478.6
OBG-58	515.4	10ft 45deg NE	28.3	NN	NN	NN	11.0	504.4
OBG-59	509.9	OL	28.1	0.0	26.0	1.0	6.0	503.9
OBG-60	508.1	OL	32.1	2.0	NN	10.3	7.5	500.6
OBG-61	505.5	GPS Location	36.1	0.0	10.3	0.0	26.0	479.5
OBG-62	502.1	GPS Location	28.5	0.2	18.0	5.2	NN	-
OBG-63	499.9	GPS Location	31.1	NN	NN	NN	NN	-
OBG-64	508.2	33ft 38deg NE	36.1	NN	28.0	20.0	NN	-
OBG-65	508.7	OL	36.2	19.5	34.0	18.0	36.0	472.7
OBG-66	512.7	GPS Location	42.1	NN	39.0	NN	40.8	471.9

Boring Log Summary Table

OBG Boring Number	Existing Elevation	Boring Location	Depth of Boring	Water Table			Rock Core	
				While Drilling	Before Casing Removed	After Casing Removed	Depth to Top of Weathered Rock	Elev of Top of Rock
OBG-67	501.4	OL	30.3	7.0	7.0	3.0	18.0	483.4
OBG-68	496.2	OL	32.3	15.0	14.5	NN	26.0	470.2
OBG-69	498.8	6.5ft 40deg NE	44.3	22.5	26.5	17.0	36.0	462.8
OBG-70	492.9	4ft 253deg SW	32.9	NN	NN	17.5	NN	-
OBG-71	487.3	OL	29.2	NN	NN	NN	14.0	473.3
OBG-72	490.6	OL	30.0	5.5	NN	4.0	NN	-
OBG-73	481.6	12ft 213deg SW	30.0	NN	NN	17.0	NN	-
OBG-74	483.8	18ft SW	30.0	9.5	NN	NN	NN	-
OBG-75	478.8	OL	29.8	NN	NN	NN	NN	-
OBG-76	464.0	26ft 88deg E	28.3	NN	NN	NN	26.0	438.0
OBG-77	466.3	GPS Location	28.7	12.5	23.5	NN	26.8	439.5
OBG-78	474.1	OL	30.0	16.0	26.0	15.0	NN	-

Notes: NN - None Noted
OL - Original Location

Rock Core Summary

B-28

Run No.	Depth (ft)		Recovery		R.Q.D		Drilling Time
	From	To	in	%	in	%	min
1	24.8	25.6	8/10	80.0	0/10	0.0	0.5
2	25.6	30.6	60/60	100.0	20/60	33.3	4.25
3	30.6	35.3	54/56	90.0	42/60	70.0	3.75
4	35.3	40.3	60/60	100.0	46/60	76.7	3.5
5	40.3	45.0	56/56	100.0	49/56	87.5	3.5
6	45.0	50.0	60/60	100.0	42/60	70.0	3.75
7	50.0	55.0	60/60	100.0	31/60	51.7	3.5

B-36

Run No.	Depth (ft)		Recovery		R.Q.D		Drilling Time
	From	To	in	%	in	%	min
1	19.7	24.7	60/60	100.0	16/60	26.7	4.5
2	24.7	29.7	58/60	96.7	7/60	11.7	3.5
3	29.7	34.7	60/60	100.0	31/60	51.7	3.5
4	34.7	39.7	58/60	96.7	40/60	66.7	3.75
5	39.7	44.7	55/60	91.7	27/60	45.0	3.5
6	44.7	49.7	60/60	100.0	50/60	83.3	3.5

B-43

Run No.	Depth (ft)		Recovery		R.Q.D		Drilling Time
	From	To	in	%	in	%	min
1	29.0	30.3	13/16	81.3	0/60	0.0	1.25
2	30.3	35.3	60/60	100.0	26/60	43.3	3.75
3	35.3	40.3	60/60	100.0	22/60	36.7	4.0
4	40.3	45.3	60/60	100.0	55/60	91.7	4.0
5	45.3	50.3	60/60	100.0	60/60	100.0	3.75
6	50.3	55.3	60/60	100.0	31/60	51.7	4.0
7	55.3	60.0	53/56	94.6	54/60	90.0	3.5

B-53

Run No.	Depth (ft)		Recovery		R.Q.D		Drilling Time
	From	To	in	%	in	%	min
1	40.0	40.75	8/9	88.9	0/9	0.0	<1
2	40.75	45.75	60/60	100.0	35/60	58.3	5
3	45.75	50.75	59/60	98.3	43/60	71.7	4.5
4	50.75	55.75	60/60	100.0	47/60	78.3	4.25
5	55.75	60.75	56/60	93.3	48.5/60	80.8	4.25
6	60.75	65.4	60/60	100.0	57/60	95.0	4.5
7	65.4	70.0	56/56	100.0	56/56	100.0	3.5