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Geotechnical Evaluation Report for Proposed Queen City Landing Project
975 Fuhrman Boulevard
Buffalo, New York

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1.00 INTRODUCTION

1.10 GENERAL

This report summarizes the results of a subsurface exploration program and a geotechnical engineering evaluation completed by Empire Geo-Services, Inc. (Empire) for the proposed Queen City Landing residential apartment / mixed use building and an adjoining parking ramp structure, planned at 975 Fuhrman Boulevard in Buffalo, New York. The approximate location of the project site is shown on Figure 1.

Trautman Associates (Trautman) retained Empire Geo-Services, Inc. (Empire) to complete a supplemental subsurface exploration and an evaluation to characterize the soil, bedrock and groundwater conditions present at the site and provided geotechnical engineering considerations and recommendations to assist Trautman and Tredo Engineers (Structural Engineering Consultant) with the design of proposed building and parking ramp structure foundations. This work was completed in accordance with our proposal dated March 11, 2016.

The supplemental subsurface exploration completed by Empire consisted of four (4) test borings drilled at the project site in March 2016. SJB Services, Inc. (SJB), our affiliated drilling company, completed the supplemental test borings for the subsurface exploration program. In addition, SJB performed laboratory testing on selected representative soil and bedrock samples to aid in our geotechnical evaluation.

These explorations are in addition to eight (8) environmental test borings and associated groundwater monitoring wells completed C&S Engineers, Inc. (C&S) in March 2016, and two (2) originally completed test borings performed for Trautman in December 2011 for the originally proposed Freezer Queen Building renovation project (Empire Geotechnical Evaluation Report, dated January 6, 2012).

Based on the findings from these exploration programs, Empire prepared this report, which summarizes the subsurface conditions encountered, and presents geotechnical considerations and recommendations for planning, design and construction of the proposed residential apartment / mixed use building and an adjoining parking ramp structure development on the site, as well as the site access drives and parking areas.

1.20 SITE DESCRIPTION

The proposed residential apartment / mixed use building and an adjoining parking ramp structure development are planned on the site of the former Freezer Queen foods manufacturing building, which is planned for demolition and removal for the Queen City Landing project development. The site is located at 975 Fuhrman Boulevard within the City of Buffalo. The site is bordered by the Buffalo Small Boat Harbor to the south, Lake Erie to the west, an existing commercial boat slip and storage yard to the north and Fuhrman Boulevard and the NYS Route 5 complex to the east. An aerial photograph of the site is presented in Appendix A.

The former Freezer Queen foods manufacturing building is supported on a pile foundation system, presumed to be timber piles. The west potion of the former Freezer Queen foods manufacturing building has previously been demolished and the existing pile caps are exposed in this area. Foundation plans prepared in 1927 for the Freezer Queen building are presented in Appendix B.

Based on the site setting conditions, as well as the significant amount of man-placed fill encountered in the test borings, it is apparent the site was originally part of Lake Erie and was reclaimed with various man-placed fill, contained within the existing marine bulkheads, to establish the current site grades. The current topography of the site, outside the existing building areas, is relatively flat, with relative ground surface elevations (El.) at the test boring locations ranging between El. 574.2 feet and El. 581.0 feet.

1.30 PROPOSED DEVELOPMENT

The proposed Queen City Landing development project is planned to include a 23-story high rise residential apartment and mixed use building, and an adjoining 4-story parking ramp structure. The residential apartment / mixed use building structure will consist of steel frame construction, while the parking ramp structure will consist of pre-cast concrete. No basements or depressed below grade parking levels are planned, however a connecting utility tunnel will be constructed with its invert near the current site grades.

The ground floor for both the apartment / mixed use building and the parking ramp structure will be established at elevation (El.) 583 feet, such that it is above the potential flood elevation of the site. This will require additional filling of the site, to raise the proposed finish grades as much as about 7 feet ±.

Tredo Engineers has indicted that maximum column loads for the building structure could be in the range of around 2,000 kips for interior columns and 1,500 kips for exterior columns. Maximum column loads for the parking ramp structure are expected to be in the range of around 1,000 kips and maximum wall loads are expected to be around 60 kips per linear foot.

Due to the extensive amount of existing fill and the known soft soil deposits present in the area of the site, as confirmed by the test borings, along with the anticipated heavy foundation loads, both the building and parking ramp structures are expected to be supported on a deep foundation system bearing on bedrock.

Surface asphalt concrete pavement parking and access drive areas will also be included as part of the project development. Traffic is expected to consist mainly of automobile/SUV's, with occasional delivery trucks.

Figure No. 2 presents a plan showing the current site conditions and proposed locations of the Queen City Landing building and parking ramp structures, along with the approximate locations of the test borings and groundwater monitoring wells.

2.00 SUBSURFACE EXPLORATIONS

2.10 GENERAL

As stated above, eight (8) environmental test borings and associated groundwater monitoring wells were completed for this project by C&S in March 2016. These test borings / monitoring wells are designated as MW-1 through MW-8 and their approximate locations are shown on Figure 2.

Two of these test borings (MW-6 and MW-7) were advanced to bedrock refusal, encountered at depths of 70.0 feet and 72.5 feet respectively. The remaining test borings were advanced to depths of 16 feet to 20 feet, and then terminated. Logs for these test borings are presented in Appendix C1.

Groundwater levels were also measured in the monitoring wells by C&S, and the data provided to Empire, for inclusion with this report. This data is also presented in Appendix C1.

Two (2) originally completed test borings were also performed by Empire / SJB for Trautman in December 2011 for the originally proposed Freezer Queen Building renovation project (Empire Geotechnical Evaluation Report, dated January 6, 2012). These borings are designated as B-1 (2011) and B-2 (2011) and their approximate locations are shown on Figure 2.

Both of these test borings were advanced to bedrock refusal encountered at depths of 71.2 feet and 75.8 feet, respectively. In addition, 5 feet of bedrock was cored in both of these borings. Logs for these test borings are presented in Appendix C2.

2.20 SUPPLEMENTAL TEST BORINGS

In addition to the above explorations, four (4) supplemental geotechnical test borings were also completed by Empire / SJB for the project in March 2016. These test borings are designated as B-1 through B-4 and their approximate "as-drilled" locations are shown on Figure 2.

The supplemental test boring locations were initially established on a site plan provided by Tredo Engineers. Using the site plan, SJB then established the gps coordinates of the test borings and laid them out in the field using a hand held gps instrument. Laser survey level techniques were utilized to determine the existing ground surface elevations at these boring locations. The ground surface elevations were referenced to the rim of an existing catch basin (benchmark used by SJB) located northeast of 2016 boring B-1, north of the existing Freezer Queen building, as shown on Figure 2. The benchmark has a reported El. of 576.49 feet, as established by others and shown on the site plan provided by Tredo.

The supplemental test borings were advanced using a Central Mine Equipment (CME) model 75 truck mounted drill rig. The test borings were advanced in the fill and indigenous overburden using hollow stem auger and split spoon sampling techniques. Continuous split spoon soil sampling and Standard Penetration Tests (SPTs) were performed to a depth of 22 feet at each of these boring locations to define the depth and characteristics of the upper fill. Below a depth of 22 feet, interval soil sampling (i.e. at intervals of 5 feet or less) was performed until auger refusal (presumed bedrock refusal) was met at depths ranging between 71.1 feet and 75.5 feet. Ten (10) feet of rock coring was performed in borings B-2 and B-3, after auger refusal was met.

The split spoon sampling and SPTs were completed in general accordance with *ASTM D 1586 - "Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils"*. Bedrock was cored using a NQ size double tube core barrel in accordance with *ASTM D 2113 – "Standard Practice for Rock Core Drilling and Sampling of Rock for Site Investigation"*.

In addition to the split spoon soil samples, two (2) relatively undisturbed Shelby tube samples were obtained, from the soft clay soils in test boring B-1 (ST-1, 32'-34') and (ST-2, 43'-45') for laboratory consolidation testing. The Shelby tube samples were obtained in general accordance with *ASTM D 1587 - "Standard Practice for Thin-Walled Tube Geotechnical Sampling of Soils"*.

A geologist from SJB prepared the supplemental test boring logs based on visual observation of the recovered soil samples and review of the driller's field notes. The soil samples were described based on a visual/manual estimation of the grain size distribution, along with characteristics such as color, relative density, consistency, moisture, etc. In addition the Unified Soil Classification System (USCS) group symbols were also established and are presented on the logs for the soil types encountered. The recovered rock core from borings B-2 and B-3 were also described, including characteristics such as color, rock type, hardness, weathering, bedding thickness, core recovery and rock quality designation (RQD). The supplemental test boring logs are presented in Appendix C3, along with general information and a key of terms and symbols used to prepare these logs.

2.30 LABORATORY TESTING

Several of the collected soil and bedrock samples were tested in SJB's geotechnical testing laboratory to confirm soil classifications, provide soil index properties, and assist with estimating soil and bedrock engineering properties. In addition, several composite soil samples, consisting of the on-site fill, were prepared and tested by SJB and Alpha Analytical to evaluate their potential corrosiveness to steel and concrete.

Geotechnical Laboratory Testing:

The geotechnical laboratory testing completed by SJB on some of the collected soil and bedrock samples included the following tests.

- Natural moisture content in accordance with *ASTM D 2216 – "Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass"*;
- Grain size analyses (sieve analyses only) in accordance with *ASTM C 136 – "Standard Test Method for Particle-Size Analysis of Soils"*;
- Liquid Limit, Plastic Limit, and Plasticity Index of Soil in accordance with *ASTM D 4318 – "Standard Test Method for Liquid Limit, Plastic Limit and Plasticity Index of Soils"*;

- Consolidation testing of undisturbed portions of the clay soil extracted from the Shelby tube samples (boring B-4, ST-1, 32' - 34' and boring B-4, ST-2, 43' - 45') in accordance with *ASTM D 2435 – "Standard Test Method for One-Dimensional Consolidation Properties of Soils"*; and

- Rock core unconfined compressive strength in accordance with *ASTM D2938-"Standard Test Method for Unconfined Compressive Strength of Intact Rock Core Specimens"*.

Analytical Laboratory Testing:

Composite samples were prepared from the fill soil samples obtained from test borings B-1, B-2 and B-4, and were tested to evaluate their potential corrosiveness to steel and concrete using the following tests.

- Resistivity, redox, pH, moisture, and sulfides according to procedures established by the Ductile Iron Pipe Research Association (DIPRA test) to provide an indication of the corrosion potential of the on-site soils with regard to buried metallic conduits (Testing completed by SJB); and

- Sulfate and chloride concentration in the soils, in accordance with EPA SW-846, with regard to potential soil impacts on buried concrete structures (Testing performed by Alpha Analytical).

The actual soil samples and bedrock core tested for the above properties, along with the test results, are summarized on Table 1. With the exception of the Sulfate and Chloride concentration tests*, the associated test data is presented in Appendix D.

** The analytical laboratory testing work for the Sulfate and Chloride concentrations in soil (being performed by Alpha Analytical) was still in progress at the time of this report. Accordingly, this laboratory test data will be summarized and submitted under a separate addendum letter when complete.*

3.00 SUBSURFACE CONDITIONS

3.10 GENERAL

The general stratigraphy encountered by the test borings consisted of topsoil and asphalt concrete pavement at the surface, along with man-placed fill extending to depths ranging between 10 feet and 19 feet, which are underlain by indigenous glacial fluvial and glacial lacustrine deposits of gravel, sand, clayey silt and silty clay soils. The deeper glacial lacustrine deposits (soft to very soft clays) generally extend to, or to slightly above bedrock, with a relatively thin stratum (i.e. typically about 2 to 7 feet or less) of denser glacial till deposited soil (mixed silty clay, sand and gravel soil) overlying Limestone bedrock. Apparent bedrock was encountered in the deeper test borings at depths ranging between about 70.0 feet and 75.8 feet, based on the auger refusal conditions encountered.

The soil and bedrock stratigraphy encountered and the groundwater conditions observed are described in more detail in the following sections and on the test boring logs in Appendix C. Also included is Table 2, summarizing the surface topsoil and asphalt concrete thicknesses where encountered, the depths and bottom elevation of the man-placed fill, the depth and elevation of apparent bedrock, and the groundwater levels observed during drilling and in the C&S groundwater monitoring wells.

3.20 SURFACE MATERIALS AND FILL SOILS

Topsoil was present at the surface of test boring locations MW-1, MW-2, MW-5 and MW-6. The thickness of the topsoil measured at these locations typically ranged between about 1-inch and 4-inches. Asphalt concrete pavement was present at the surface of the 20011 test borings B-1 and B-2, and at the 2016 test borings B-1, B-2, B-3, MW-3, MW-4 and MW-7. The thickness of the asphalt concrete at these locations ranged between 3-inches and 6-inches. Within 2016 test boring B-2 a 6-inch concrete slab was present beneath the asphalt pavement.

The surface material (topsoil and asphalt concrete) thickness measurements are widely spaced, are based on the driller's interpretation, and are approximate. Accordingly, these measurements should not be solely relied on for accurate construction quantity estimates. We recommend the Contractor, and/or others, make their own observations and measurements, prior to bidding and construction, to determine the quantities, costs and efforts that will be required for topsoil and asphalt concrete removal and associated replacement with appropriate suitable fill materials. In addition, both exposed and buried foundations, structures, slabs and utilities are expected to be present on the site, and therefore should be anticipated by the design team and Contractor.

Beneath the topsoil and pavement, and from the surface of the remaining boring locations, man placed fill soils consisting of various shades of brown, gray, red and black silty clay, clayey silt, sand and gravel soils, along with occasional zones and varying amounts / inclusions of intermixed brick, slag, concrete, cinders, asphalt, coal fragments, and organics were encountered, in each of the test borings.

The man-placed fill soils and materials were found to extend to depths ranging between about 10 feet and 19 feet at the test boring locations. The depths and bottom of fill elevations encountered at the test boring locations are summarized on Table 2.

Based on it's varying composition, the fill appears was placed in a generally random and un-controlled manner. In addition, Standard Penetration Test (SPT) "N' values obtained in the fill were highly variable with both location and depth, indicating the fill was generally not densified in a controlled manner at the time of it's placement.

Based on the site conditions and extensive amount of fill present is apparent the site was originally part of Lake Erie and was reclaimed with the various man-placed fill, to establish the current site grades. It can be expected that man-placed fill materials / soils will also be present, and will extend to the bottom of previous excavations made for construction of both existing and former structures and utility lines present within the site. Existing foundation / buried structure obstructions, as well as debris and/or large or significant rubble or concrete obstructions, generally did not appear to be encountered during the advancement of the test borings.

3.30 INDIGENOUS SOILS

Beneath the fill, indigenous soils consisting of various shades of brown, black, and gray glacial fluvial and glacial lacustrine soil deposits of sand, silt, clayey silt, and silty clay were generally encountered, which extend to, or near the top of bedrock. These deposits are classified as SP, SP-SW, ML, CL, and CL-CH group soils using the Unified Soil Classification System (USCS).

At some of the test boring locations a relatively thin stratum (i.e. typically about 2 to 7 feet or less) of denser glacial till deposited soils (mixed silty clay, sand and gravel soil) was encountered just above bedrock. These soils are classified as GC-GM and SM group soils using the USCS.

Standard Penetration Test (SPT) "N" values obtained in the cohesive fine grained silty clay and clayey silt soil stratums ranged from 23 to "woh - weight of hammer" (i.e. the sample spoon was advanced with only the weight of the drop hammer and drill rods applied statically to the sample spoon), indicating the consistency of the cohesive soils varies from very stiff to very soft, however typically they are medium to very soft. SPT "N" values obtained in the more granular sand and clayey silty sand and gravel soil deposits, and in the non-plastic silt soil deposits, ranged from

38 to 2 indicating the relative density of these soils varies from compact to very loose, however typically they are firm to very loose.

The silty clay soil deposits were generally encountered beginning at depths ranging from about 18 feet to 26 feet, with the medium to very soft consistency clay being encountered at a depth of around 25 feet and extending to or near the top of bedrock. The moisture content, liquid limit, plastic limit, and plasticity index testing on the split spoon and Shelby tube samples obtained from the silty clay soils, as summarized on Table 1, indicate their moisture contents range from 18.7% to 42.3% and their plasticity indices range between 8 and 21, indicating they are of a low to high plasticity.

The consolidation testing indicates that the very soft consistency clay soils are normally to somewhat under-consolidated having estimated pre-consolidation pressures ranging from about 0.8 tons per square foot (tsf) to 1.1 tsf. Accordingly, consolidation settlement within the soft clay stratum is expected to be generally in virgin compression, based on the anticipated foundation and site filling loads. The modified recompression indices ($C_r/1+e_0$) of these samples range from about 0.007 to 0.016 and the modified compression indices ($C_c/1+e_0$) range from about 0.062 to 0.120.

3.40 REFUSAL CONDITIONS AND BEDROCK

Auger refusal (presumed bedrock refusal) was encountered in the 2011 test borings B-1 and B-2, and in the 2016 test borings B-1 through B-4, MW-6 and MW-7 at depths ranging between about 70.0 feet (boring MW-6) and 75.8 feet (2011 boring B-2). The depth and elevation where auger refusal was met at each of these borings is summarized on Table 2.

It appears the auger refusal material encountered was generally bedrock. However, it is also possible that a cobble or boulder, could also have resulted in the refusal at the locations where coring was not performed to confirm the nature of the refusal material.

Presuming the auger refusal encountered at each of these locations is actually bedrock, it appears the bedrock elevation ranges from about El. 506.7 feet at 2016 boring B-3 to El. 498.9 feet at 2016 boring B-4, indicating the top of bedrock elevation drops about 8 feet from east to west across the proposed building and parking ramp site.

Bedrock core was obtained in 2011 test borings B-1 and B-2, and in the 2016 test borings B-2 and B-3, after reaching auger refusal. The bedrock core recovered consisted of gray, medium hard to hard, sound, laminated to thickly bedded Limestone. The rock core recoveries ranged from 92% to 100% and the rock quality designation (RQD) values ranged from 85% to 98% indicating the recovered cores have a "good" to "excellent" rock mass quality. Geologic maps indicate the uppermost bedrock formation in this area of the City of Buffalo is the Middle Devonian Period, Onondaga Limestone geologic formation.

The geotechnical laboratory testing completed on the recovered bedrock core specimens from 2016 borings B-2 and B-3, as summarized on Table 1, indicates the bedrock core tested has unconfined compressive strengths of 15,060 psi and 16,600 psi.

3.50 GROUNDWATER CONDITIONS

Water level measurements were made in the test borings during and at the completion of drilling and sampling, as noted on the test boring logas. As summarized on Table 1, freestanding water was observed in the test borings during drilling at depths ranging between 5 feet and 14 feet. These levels correspond to elevations ranging between El. 575.0 feet and El. 562.4 feet.

In some cases it appears groundwater may not have sufficient time to accumulate and fully stabilize in the boring holes within the time period that had elapsed from the noted drilling operation phase and the time of measurement.

Water levels were also measured by C&S between March 17^{th} and 23^{rd}, 2016, in the groundwater monitoring wells MW-1 through MW-8. These measurements are summarized on Table 2. These measurements indicated stabilized groundwater levels ranged between a depth of 2.5 feet at MW-7 and 7.6 feet at MW-1 corresponding to elevations ranging between El. 575.7 feet at MW-2 and El. 571.0 feet at MW-6.

It is also possible that some perched or trapped groundwater could also be present in the upper more permeable fill soils, which overlie less permeable fill soils. Perched groundwater conditions can be particularly prevalent during and following heavy or extended periods of precipitation and during seasonally wet periods.

Lake Erie is also prone to a seiche effect from a strong sustained wind event out of the southwest. During these events the water levels in the northeastern end of the lake can rise several feet. Historical these water levels have been measured as high as El. 576 to 577 feet. Accordingly, these fluctuations can also occur in the groundwater levels along the adjacent shorelines.

It is noted that a Lake Erie seiche event occurred on March 28, 2016, after SJB had just completed test boring B-4. The driller noted the area around boring B-4 had flooded with about 2 to 3 feet of water, as the result of high winds out of the southwest, causing the lake levels to rise. Data obtained from the NOAA web site: http://tidesandcurrents.noaa.gov/waterlevels.html indicated a reported Lake Erie water level of El. 576.1 feet at 1 pm on March 28, 2016, at the Buffalo recording station.

It should be expected that perched and permanent groundwater conditions could vary with changes in soil conditions, precipitation and seasonal conditions, as well as with fluctuations in the Lake Erie levels.

3.60 SOIL CORROSION POTENTIAL

DIPRA Corrosion Potential:

Three (3) composite soil samples were prepared from the samples obtained at 2016 test boring locations B-1, B-2 and B-4, consisting of the on-site fill. The composite samples were tested for resistivity, redox, pH, sulfides, and moisture according to procedures established by the Ductile Iron Pipe Research Association (DIPRA).

This analytical laboratory test data is included in Appendix D and the DIPRA point values obtained are also summarized on Table 1. The total DIPRA points ranged between 6 and 11.

Based on the DIPRA publication "American National Standard for Polyethylene Encasement for Ductile Iron Pipe Systems", if the total DIPRA points exceed 10, the soil is considered corrosive to ductile iron pipe, and protection against exterior corrosion should be provided. Accordingly, based on the test results, it is recommended that buried metallic pipes and conduits be provided with a suitable protective coating or cathodic protection to resist potential corrosion.

Sulfate Attack Potential:

The composite samples prepared from 2016 test borings B-1, B-2 and B-4 were also tested for chlorides and sulfates.

The analytical laboratory testing work for the Sulfate and Chloride concentrations (being performed by Alpha Analytical) was still in progress at the time of this report. Accordingly, this laboratory test data will be summarized and submitted under a separate addendum letter when complete.

4.00 GEOTECHNICAL CONSIDERATIONS AND RECOMMENDATIONS FOR APARTMENT / MIXED USE BUILDING AND PARKING RAMP STRUCTURE DEVELOPMENT

4.10 GENERAL CONSIDERATIONS

Foundation support of the proposed apartment / mixed use building and parking ramp structure will be impacted primarily by both the significant amount of existing uncontrolled fill and the deeper medium to very soft silty soil deposits present. In addition existing foundation/structure elements present are also expected to have impacts on the site preparation and foundation installation.

The existing man-placed uncontrolled fill encountered extended to depths ranging between about 10 feet and 19 feet, at the test boring locations. The medium to very soft and loose to very loose indigenous soils were encountered below the fill and were found to extend to or near the top of bedrock, which was present at depths ranging between about 70.0 feet and 75.8 feet below the existing ground surface.

The existing fill and underlying soft clay soils have very marginal bearing capacity support and would be susceptible to excessive total and differential settlement of a conventional spread or mat type foundation system. Therefore, the in-situ soil conditions are not considered suitable for the use of spread or mat type foundations to support of the proposed building and parking ramp structures. Accordingly, it is recommended the proposed structures be supported on a driven pile foundation system bearing on the Limestone bedrock.

In addition it is also recommended the at grade floor systems for both the building structure and the parking ramp structure, be designed as pile supported structural slabs, due to the expected settlement from the additional site filling. The potential settlement from the site filling will also warrant that the underground utility tunnel be pile supported.

The raising of site grades as much as 7 feet within the building and parking ramp areas, to establish the at-grade ground level finished floor elevation of 583.0 feet, is expected to result in excessive post construction settlement beneath the additional fill. Based on our estimates, the weight of the additional fill could potentially result in approximately 4-inches ± of long term consolidation settlement, primarily occurring within the soft to very soft clay soils. Due to the thickness of this highly compressible stratum, it is estimated that a period of around 3 to 7 years would be required for approximately 75% of the consolidation settlement to occur.

Incorporating the use of light weight "Geofoam" material in the site filling, to reduce the fill load, would normally be a consideration to mitigate this condition. However, due to the possible flooding, the buoyancy of this material will make this option unsuitable.

The expected settlement within the soft to very soft clay soils will also need to be taken into account when sizing the pile foundation system. The expected settlement is expected to induce down drag or negative skin friction forces on the piles. Accordingly, the allowable design capacity of the piles recommended below have been reduced to account for these conditions.

The site preparation work will need to consider the presence of existing foundation elements (pile caps, grade beams and piles), slabs, and underground structures and utilities, which are present from the former Freezer Queen facility and previous site development. Excavation and removal, or drilling through these components will be required for installation and construction of the new foundation system components as well construction of new underground utility tunnel. In addition, it is recommended that existing piles be extracted, where they may interfere with installation of the new piles. As a minimum, existing piles should be cut off and removed to below the new pile caps, grade beams and structural floors.

More detailed recommendations to assist in planning and design of the building and parking ramp foundations, and associated site development are provided in the following report sections.

4.20 DRIVEN PILE FOUNDATION DESIGN

4.20.1 General:

Limestone bedrock, which appears was encountered at depths ranging between about 70.0 feet and 75.8 feet below the existing ground surface at the deep test boring locations (i.e. corresponding to elevations ranging between about El. 506.7 feet and El. 498.9 feet), will provide a suitable bearing stratum for a driven pile foundation system. H-piles or pipe piles driven to refusal on the bedrock will derive their capacity predominately through end bearing.

All driven piles should be of a minimum Grade 50 (Fy ≥ 50 ksi) steel. Consideration should be given to equipping the piles with a hardened driving tip or shoe to limit potential damage when driving through the upper fill and into the bedrock.

Pipe piles should have a minimum wall thickness of at least 0.375 inches and may be driven open ended or with a closed end, as determined appropriate by the pile driving contractor. Pipe piles driven open ended should be equipped with a flush inside mounted driving shoe to limit potential damage when driving through the upper fill and into the bedrock.

If a closed end pipe pile is used, a flush steel plate tip, at least 0.50 inches thick, should be welded to the pile tip to form the closed end. Following driving and acceptance, the annulus of the closed end pipe pile can be filled with concrete or pea gravel, as determined appropriate.

The use of "Mill 2nds" pipe piles (i.e. off specification oil / gas field pipe) will be acceptable provided these pipe products meet the minimum yield strength used for design, and that appropriate mill certifications are provided by the Contractor / Supplier.

Driven piles should be spaced a minimum of 3 pile widths apart, or three feet, whichever is greater. Exterior pile caps and grade beams should be embedded a minimum of 4 feet below final exterior grades for frost protection.

Possible existing foundation /structure elements, buried slabs, etc., along with any zones of rubble, and possible boulder size slag obstructions, which may be present / buried within the existing fill should be anticipated by the pile driving contractor. Therefore, pre-drilling, pre-excavation and/or existing pile extraction may be necessary at some of the pile foundation locations, in order to effectively locate and drive the piles.

4.20.2 Axial Compressive Capacities:

An H-pile or pipe pile, driven to refusal on the Limestone bedrock, may be designed for an allowable axial compressive capacity equal to 35% of the pile yield strength or a maximum of 17.5 kips per square inch (ksi), whichever is less, times the effective cross sectional area of the pile. A 10% reduction in the cross sectional area should be considered to account for potential corrosion / section loss over the pile life.

The expected settlement within the existing fill and indigenous soils, particularly the soft to very soft clay soils, due to the site filling, will also need to be taken into account when sizing the selected piles for the foundation system. The expected settlement is expected to induce down drag or negative skin friction forces on the piles. Accordingly, it is recommended the allowable design capacity of the piles also be reduced by an additional 25% to account for these conditions.

The following table summarizes the allowable axial compressive capacity and required ultimate test capacity for three (3) types of H-pile sections based on the above design criteria. These capacities assume the use of Grade 50 Steel, as well as account for the recommended 10% section loss for corrosion and an additional 25% to account for potential down drag forces.

Pile Section	Allowable Axial Compressive Capacity per Pile, including Corrosion and Down Drag	Required Ultimate Test Capacity
HP 12 x 53	92 tons	272 tons
HP 12 x 74	129 tons	382 tons
HP 14 x 89	154 tons	457 tons

The following table summarizes the allowable axial compressive capacity and required ultimate test capacity for three (3) types of pipe pile sections based on the above design criteria. These capacities also assume the use of Grade 50 Steel, as well as account for the recommended 10% section loss for corrosion and an additional 25% to account for potential down drag forces.

Pipe Pile Section	Allowable Axial Compressive Capacity per Pile, including Corrosion and Down Drag	Required Ultimate Load Test Capacity
10.750" O.D. Pipe Pile (0.375" Wall Thickness)	72 tons	214 tons
12.750" O.D. Pipe Pile (0.375" Wall Thickness)	86 tons	255 tons
14.000" O.D. Pipe Pile (0.375" Wall Thickness)	95 tons	281 tons

The ultimate load test capacities presented above assume a Factor of Safety of 2.0 as required by the Building Code of New York State, as well as consider the section reduction for potential corrosion loss and the 25% reduction in the allowable capacity due to down drag.

The above H-pile and pipe pile sections can be considered for use, based on the actual structure loads and design conditions/requirements. Other H-pile or pipe pile sections can also be used, based on Contractor recommendations and current product availability. The allowable capacities and required ultimate load test capacities, for alternative pile sections, should be computed based on the design criteria outlined above.

Driven pile foundations end bearing on the bedrock should undergo insignificant total settlement (i.e. generally limited to the elastic shortening of the pile) when designed and constructed in accordance with our recommendations.

4.20.3 Uplift Resistance:

Uplift resistance (i.e. side shear resistance) of the driven piles end bearing on the bedrock can be computed using an average allowable unit uplift side shear resistance of 85 pounds per square foot (psf), within the fill and indigenous soils.

In computing the uplift resistance, the pile length from the bottom of pile cap to the top of bedrock elevation can be used. The boxed perimeter of the H-piles or the outer circumference of the pipe pile section can be used in calculating the uplift resistance of the individual piles. The lesser of the uplift resistance of the boxed perimeter of a pile group, versus the sum of the individual pile uplift resistances within the pile group, should be used for group pile uplift resistance.

Micro piles or rock anchors, drilled and grouted into the bedrock, could also be considered to supplement the uplift resistance of the driven piles, at locations which require additional resistance of the uplift loads. Empire can provide recommendations for micro piles / rock anchors, should they become necessary.

4.20.4 Lateral Load Resistance:

If requested, Empire can perform pile lateral load – deflection analyses for lateral loads applied to the top of the pile. Depending on the pile spacing and load orientation between piles within a group, the total lateral resistance capacity of the individual piles within the group may not be fully developed. Accordingly, it is recommended that the lateral analyses be performed on a case specific basis as the design is finalized and the actual pile sections and arrangements are selected. For these analyses, Empire would need the pile type and arrangement (number of piles, orientation, and spacing), vertical, lateral and moment loads applied to the top of the piles, and the conditions in which the pile / pile cap interact (i.e. free head vs. fixed head condition).

4.20.5 Load Testing:

A pile load testing program should be performed prior to installation of the production piles to confirm the design loads will be achieved.

At least 3 random piles of each driven pile type used, or no less than a total of 10 piles, should be dynamically tested in accordance with *ASTM D 4945 – "Standard Test Method for High Strain Dynamic Testing of Piles"* to confirm that the pile compressive capacity has been obtained with an adequate factor of safety (i.e. Factor of Safety of 2.0 or greater as required by the Building Code of New York State). Dynamic testing should also be performed on any piles which are suspect of not having been seated on bedrock.

For driven piles subject to uplift loads, at least 1 pile should be tested in accordance with *ASTM D 3689 – "Standard Test Method for Individual Piles Under Static Axial Tensile Load"* to confirm the that the design uplift capacity has been obtained with an adequate factor of safety (i.e. Factor of Safety of 2.0 or greater). Acceptance of the uplift load test can be based on the following criteria: "The total net uplift of the pile, after rebound, should not exceed 0.01-inches per ton of the test load (i.e. twice the design load) or 0.250-inch, whichever is less".

4.30 FLOOR CONSTRUCTION

As stated above, it is recommended the at grade floor systems for both the building structure and the parking ramp structure, be designed as pile supported structural slabs, due to the expected settlement from the additional site filling. It is recommended that a minimum 4-inch thick layer of compacted Structural Fill (Subbase Stone) material, as described in Appendix E, be placed beneath the pile supported structural floors to provide a suitable and uniform working surface to set the floor system reinforcing steel and construct the structural slab.

A suitable vapor barrier beneath the apartment / mixed use building floor system should be also considered, as appropriate and in consultation with C&S, to limit any potential odors from the existing fill from entering the building. Vapor barrier systems should be carefully detailed and constructed to seal off all potential vapor pathways, such as at floor joints and utility / floor penetrations, etc.

4.40 DEPRESSED STRUCTURE DESIGN

For design purposes we recommend the groundwater conditions be assumed at about El. 578 feet to account for potential groundwater level fluctuations, or the 100 year flood elevation, whichever is higher,

Accordingly, any below grade depressed structures (i.e. elevator pit structures, utility tunnel, etc. which would be situated below the recommended design groundwater elevation, should be designed to resist full hydrostatic pressures acting on the structure walls and bottom slab, as well as be properly waterproofed.

Where loading docks or any other slightly depressed structures are situated above the design groundwater elevation, a foundation drainage system, as discussed below, should be incorporated, to relieve hydrostatic pressures from developing against the structure walls and bottom.

Below grade depressed structure walls and earth retaining foundation walls should be designed to resist lateral earth pressures generated by the earth backfill and any temporary or permanent surcharge loads. Below grade depressed non-yielding earth retaining structure walls (i.c. loading dock walls, elevator pits, etc.), should be designed to resist "at rest" lateral earth pressures, based on the soil parameters below. Walls which are allowed to yield (i.e. cantilevered earth retaining walls) may be design on the basis of "active" lateral earth pressures. These parameters are based on the wall backfill consisting of Suitable Granular Fill or Structural Fill, as described in Appendix E.

In addition, if the structure is designed for full hydrostatic pressures, the walls should be designed to resist the hydrostatic pressures as well as the lateral earth pressures acting the walls. In this case, the lateral earth pressure should be computed based on a submerged soil unit weight below the design groundwater level. In addition, the floor / bottom slab must be designed to resist the hydrostatic uplift pressure acting on it. In this case, the depressed structure should also be fully water proofed.

Recommended Soil Parameters for Depressed Structure Foundation Wall Design

- Coefficient of "At-Rest" Lateral Earth Pressure – 0.47
- Coefficient of "Active" Lateral Earth Pressure – 0.31
- Coefficient of "Passive" Lateral Earth Pressure – 3.25
- Angle of Internal Friction – 32 Degrees

- Moist Unit Weight of Soil – 135 pcf
- Submerged or Buoyant Unit Weight of Soil – 73 pcf
- Surcharge Load Lateral Coefficient – 0.50

Accordingly, the following Equivalent Fluid Unit Weights for Soil Lateral Earth Pressure are derived from the above lateral earth pressure coefficients and soil unit weights.

Lateral Earth Pressure Type	Equivalent Fluid Unit Weight for Moist Soil (Above Groundwater)	Equivalent Fluid Unit Weight for Submerged or Bouyant Soil (Below Groundwater)
"At-Rest" Earth Pressure	64 pcf	35 pcf
"Active" Earth Pressure	42 pcf	23 pcf
"Passive" Earth Pressure	435 pcf	238 pcf

For the design of the portion of the wall below the design groundwater level, the hydrostatic pressure acting the wall must also be added to the submerged or buoyant soil equivalent fluid unit weight lateral earth pressures acting on the wall.

Perimeter foundation wall drains, to intercept perched groundwater and relieve potential hydrostatic pressures, should be provided where a below grade structure or earth retaining foundation wall is situated above the design groundwater elevation. The foundation drainage system must be properly designed, installed and maintained for long-term performance and should include such features as clean-outs, to properly maintain the system. The cleanouts should be spaced at maximum 50 feet intervals along straight runs, and at all corners or bends The foundation drainage system should drain to a sump and pump system or suitable uninterrupted gravity relief point . The foundation drain pipes should be set at a minimum depth of 1.0 foot below the structure floor grade, or just above the lowest adjacent grade in the case of retaining walls.

The foundation drainage system should include a geotextile, selected considering drainage and filtration, installed around drainage stone surrounding a slotted under-drain pipe. The drainage stone should be sized in accordance with the pipe slotting or perforations. A crushed aggregate conforming to NYSDOT Standard Specifications Section 703-02, Size Designation No. 1 (½-inch washed gravel or stone) is generally acceptable for slotted under-drain pipe. 6-inch diameter slotted HDPE or PVC underdrain pipe is recommended. The foundation drainage stone and surrounding geotextile, along the walls, should extend above the drainpipe a minimum of 2 feet.

A suitable geosynthetic drainage composite (i.e. "Grace Hydroduct", "Miradrain", "Delta MS" or other suitable equivalent) should be placed against the foundation wall, above the drainage system, to allow infiltration to the drainage system. The drainage media against the wall should extend to about 1 to 2 feet below the finished grade surface, where it may be capped off with the foundation backfill material.

4.50 SEISMIC DESIGN

Based on the subsurface conditions encountered in the test borings, the proposed Queen City Landing apartment / mixed use building and parking ramp development site should be classified as Seismic Site Class "E" in accordance with Table 1613.5.2 of the Building Code of New York State - December 2010 (NYS Building Code). Therefore, seismic design should be based on this site classification.

The spectral response accelerations in the area of the project site (975 Fuhrman Boulevard) in the City of Buffalo, New York were obtained by Empire using the United States Geological Survey (USGS) web site application (https://geohazards.usgs.gov/secure/designmaps/us/). The accelerations are based on the 2009 NEHRP Recommended Seismic Provisions, which makes use of the 2008 USGS seismic hazard data. The uniform hazard acceleration values obtained from this application were then adjusted, as recommended by the USGS, to obtain the 2% probability in 50 years mapped geometric mean accelerations, as presented in the NYS Building Code.

The calculated geometric mean spectral response accelerations for Site Class "B" soils are 0.214g for the short period (0.2 second) response (S_S) and 0.050g for the one second response (S_1). For design purposes, these spectral response accelerations were then adjusted for the Seismic Site Class "E" soil profile determined for the project site.

Accordingly, the adjusted spectral response accelerations for Site Class "E" are as follows:

- Short Period Response (S_{MS}) - 0.535g
- 1 Second Period Response (S_{M1}) - 0.175g

The corresponding five percent damped design spectral response accelerations (S_{DS} and S_{D1}) are as follows:

- S_{DS} - 0.357g
- S_{D1} - 0.117g

4.60 PAVEMENT DESIGN

4.60.1 Flexible Asphalt Pavement Design:

Flexible asphalt pavement design recommendations are provided for both a Heavy Duty Pavement (i.e. for use in the main entrance access drive, building access roads and truck delivery areas) and for a Light Duty Pavement (i.e. for use in the automobile only parking lot areas).

Heavy Duty Asphalt Concrete Pavement:

- 1.5 inches – Top Course
- 3.0 inches – Binder Course
- 14 inches – Subbase Course*
- Geotextile
- Prepared Subgrade

Light Duty Asphalt Concrete Pavement:

- 1.5 inches – Top Course
- 2.0 inches – Binder Course
- 12 inches – Subbase Course*
- Geotextile
- Prepared Subgrade

*It may be necessary to increase the subbase thickness in some areas to improve subgrade conditions and to promote drainage to underdrains, etc, as discussed below.

Materials for the above flexible pavement structure components should consist of the following:

A. Asphalt Concrete Top Course - NYSDOT Standard Specifications - Hot Mix Asphalt, Type 7 F2 Top Course.

B. Asphalt Concrete Binder Course - NYSDOT Standard Specifications - Hot Mix Asphalt, Type 3 Binder Course.

C. Subbase Course – Should comply with NYSDOT Standard Specifications, Item No. 304.12 - Type 2 Subbase.

D. Geotextile - Woven polypropylene stabilization/separation geotextile (i.e., Mirafi 600X or approved suitable equivalent).

E. Prepared Subgrade – As recommended in Section 5.40

4.60.2 Pavement Drainage:

The installation of underdrains or edge drains are recommended to drain the pavement subbase course and subgrades in order to limit the potential for frost action and improve pavement structure performance and design life.

Underdrains should include a geotextile (i.e. Mirafi 160N or suitable equivalent), selected considering drainage and filtration, installed around drainage stone surrounding a slotted or perforated drain pipe. The drainage stone should be sized in accordance with the pipe slotting or perforations. A crushed aggregate conforming to NYSDOT Standard Specifications Section 703-02, Size Designation No. 1 (½-inch washed gravel or stone) is generally acceptable for slotted underdrain pipe. The underdrain pipes should be set in the bottom of the subbase layer, or preferably below the top of the soil subgrade elevation. The drainage stone and surrounding geotextile should extend above the underdrain pipe and into the subbase layer. Underdrain pipes should be connected to the site storm water drainage system.

Alternatively, the pavement subbase course should be allowed, as a minimum, to daylight/drain to an adjacent perimeter drainage swale or other drainage relief point. Accumulation of water on pavement subgrades should be avoided by grading the subgrade to a slope of at least 1 to 2 percent to allow drainage to the edge drains or drainage swale.

5.00 SITE PREPARATION AND CONSTRUCTION CONSIDERATIONS

5.10 CONSTRUCTION DEWATERING

Construction dewatering should be implemented as necessary for surface water control and for excavations, which extend below the groundwater. Surface water should be diverted away from open excavations and prevented from accumulating on exposed subgrades. The exposed fill and indigenous soil subgrades will be susceptible to strength degradation in the presence of excess moisture.

The site contains zones of highly porous fill materials (i.e. slag, brick, etc) and gravel and sand soils, which can yield substantial and unpredictable quantities of groundwater. These groundwater conditions can vary with location and depth, are difficult to quantify, and in some cases are expected to require high capacity pumps to effectively depress and control the groundwater within excavations made in these materials. The indigenous clayey silt, non-plastic silt, and fine sand soils will generally yield lesser quantities of groundwater, however, they can also be expected to undergo sidewall and bottom instability where excavations extend into these soils below the groundwater table.

Construction dewatering procedures should properly depress and maintain the groundwater levels at least 1 to 2 feet below the excavation bottoms.

It is anticipated that conventional sump and pump methods of dewatering, along with the placement of a crushed stone working mat/drainage layer and underdrains, in the bottom of the excavation can be used to control groundwater conditions, where the excavation may just encroach a few feet into the permanent groundwater surface. The working mat/drainage stone material can consist of NYSDOT Standard Specifications Section 703-02, Size Designation No. 2 or No. 3, washed crushed gravel or stone, and a surrounding separation/drainage geotextile, such as Mirafi 160 N or equivalent. Larger particle stone products are not recommended, as they may potentially hinder pile installation.

For deeper excavations below the permanent groundwater level, particularly where the existing more porous fill also extends significantly below the groundwater table, more substantial methods of dewatering are expected to be necessary. In this case, a Dewatering Specialty Contractor may need to be consulted and contracted to design an appropriate dewatering system.

It is recommended that the Contractor excavate some test pits in advance of the excavation work, particularly where deeper excavations are required, to ascertain potential groundwater conditions and plan the dewatering that will need to be implemented. Dewatering systems should be operated on a continual basis, until the excavations are backfilled several feet above the stabilized permanent groundwater level. Groundwater dewatering plans should include implementation of measures to control erosion, sedimentation and the migration of soil fines. The design of the dewatering systems and discharges should also account for potential environmental concerns associated with the on-site groundwater. In addition, dewatering system pump discharges should be monitored for potential pumping of soil fines, which should not be permitted.

5.20 DRIVEN PILE CONSTRUCTION AND TESTING

The piles should be driven to refusal, on the Limestone bedrock, using a pile hammer having a suitable energy rating. The pile driving criteria should be confirmed by the Contractor through the use of the wave equation, based on the actual pile, pile hammer and cushions that will be used, to determine the final driving criteria and that adequate stresses can be developed in the pile to confirm its capacity through dynamic testing and to determine that the pile will not be overstressed during driving. Pile stresses should not exceed 85% of the pile yield stress. Plumbness of the piles should be maintained within 1% of the total length. Any misaligned or damage piles should be replaced.

Refusal should be defined as when about 5 blows have been recorded for less than ¼ inch of pile penetration and the pile reaches the anticipated bedrock elevation.

All static and dynamic load tests should be observed and evaluated by a geotechnical Professional Engineer licensed in the State of New York, who is retained by the Owner. The static and dynamic load tests should be set up and run by the pile driving Contractor under the geotechnical Professional Engineer's observation.

Qualified geotechnical testing personnel, under the guidance and supervision of the geotechnical Professional Engineer, should observe all production pile driving and should prepare individual pile driving reports for each pile installed. The reports should include, pile number and location, hammer and cushion types, pile size and material, installed length, blows per foot, unusual conditions encountered during driving, top of pile elevation following driving and notes on any necessary re-striking, etc. The Contractor should mark all piles with appropriate foot and inch intervals in order to properly monitor and document the pile installations and testing. Installed piles should also be monitored for potential heaving during installation of adjacent piles. Any piles that heave should be re-driven and reseated as appropriate.

5.30 FOUNDATION ELEMENT EXCAVATION AND BACKFILLING

Excavations for grade beam and pile cap construction, as well as other structure excavations, should be performed using a method, which reduces disturbance to the subgrade soils. If any soils containing organics, highly voided demolition debris/rubble, or otherwise deleterious soil material are encountered, they should be removed and replaced with compacted Structural Fill or Suitable Granular Fill, as recommended in Appendix E.

Subgrades for grade beam, pile cap and structure construction should be protected from precipitation and surface water. Water should not be allowed to accumulate on the soil subgrades and the subgrades should not be allowed to freeze, either prior to or after construction of foundations. If subgrades are not protected and degrade, they should be undercut/removed accordingly.

Grade beam and pile cap excavations should be backfilled as soon as possible and prior to construction of the superstructure. It is recommended that excavations within the building area and pavement areas be backfilled with a properly compacted Structural Fill or Suitable Granular Fill material, as recommended in Appendix E.

5.40 SUBGRADE PREPARATION FOR BUILDING PAD AND PAVEMENT CONSTRUCTION

The site preparation work should be performed during seasonally dry periods to minimize potential degradation of the subgrade soils and undercuts, which may be required to establish a stable base for construction. It should be understood that the fill soils that will be exposed are potentially sensitive and may degrade and lose strength when they are wet and disturbed by construction equipment traffic. Accordingly, efforts should be made to maintain the subgrades in a dry and stable condition at all times, and not permit excessive or heavy construction traffic directly over these soils.

All existing, asphalt concrete, surface structures, trees, vegetation, topsoil, etc. and any other deleterious materials within the proposed floor and pavement areas should be removed. Existing structures should be removed to a depth of at least 2 feet below the bottom of the Structural Fill (Subbase Stone) course for the floor and pavement construction.

Following removal of the surface materials and excavation to the proposed subgrades, the exposed fill soil subgrades should be thoroughly compacted/densified and then proof-rolled. The subgrade compaction should be performed, prior to required fill placement, using a vibratory smooth drum roller weighing at least 10 tons. The roller should be operated in the vibratory mode for compacting the subgrades and in the static mode for proof rolling. The roller should complete at least four (4) passes over the exposed subgrades for the compaction/densification operation and at least two (2) passes for the proof rolling evaluation. We recommend that the exposed subgrades in deeper excavations at or near the groundwater level should not be compacted / proof-rolled as this may comprise the integrity of these subgrade soils

The subgrade compaction and proof-rolling should be done under the guidance of, and observed by, a representative of Empire. Any areas, which appear wet, loose, soft, unstable or otherwise unsuitable, should be undercut. Over excavation, which may be required as the result of the proof-rolling, should be performed based on evaluation of the conditions by Empire. Resulting over-excavations should be backfilled with compacted Structural Fill/Subbase Stone material as described in Appendix E.

Suitable Granular Fill or Structural Fill, as described in Appendix E, should be used to backfill excavations and as subgrade fill to raise site grades. Empire should be consulted regarding the acceptability of any proposed alternative subgrade fill materials, which do not meet the requirements recommended for Suitable Granular Fill or Structural Fill. All fill placement and compaction should also be closely monitored and tested on a "full-time" basis by a representative of Empire.

Efforts should be made to maintain the subgrades in a dry and stable condition at all times, and limit construction traffic directly over the subgrade soils, particularly if they become wet.

Subgrade fill should be placed to a stable condition and should not "pump", "rut" or show signs of movement or significant deflection (i.e. unstable conditions) as it is being constructed. The fill subgrades should be properly graded, drained and protected from moisture and frost. Placement of fill over wet, soft, snow covered or frozen subgrades is not acceptable. Any subgrades, including existing soil subgrades or fill subgrades, which become damaged, rutted or unstable should be undercut and repaired as necessary prior to placement of the Structural Fill or Subbase Stone courses.

5.50 PAVEMENT CONSTRUCTION

Placement of the pavement subbase stone can proceed, following proper subgrade preparation, proof-rolling and subgrade filling as described in Section 5.40. The subbase stone should be placed and compacted in accordance with the recommendations presented in Appendix E for Structural Fill. Installation of adjacent geotextile panels should have minimum overlap of 12 inches to 18 inches. Construction of the Asphalt Concrete Pavement should be performed in accordance with NYSDOT Standard Specification Section 400. In addition, placement of asphalt concrete pavement courses should not be permitted on wet or snow covered surfaces or when the subgrade surface is less than 40° F.

6.00 CONCLUDING REMARKS

This report was prepared to assist in planning, design and construction of the proposed Queen City Landing residential apartment / mixed use building and an adjoining parking ramp structure, planned at 975 Fuhrman Boulevard in the City of Buffalo, New York. The report has been prepared for the exclusive use of Trautman Associates; Tredo Engineers; and members of the design team, for specific application to this site and this project only.

The recommendations were prepared based on Empire Geo-Services, Inc.'s understanding of the proposed project, as described herein, and through the application of generally accepted soil and foundation engineering practices. No warranties, expressed or implied are made by the conclusions, opinions, recommendations or services provided.

Empire Geo-Services, Inc. should be informed of any changes to the planned construction so that it may be determined if any changes to the recommendations presented in this report are necessary. Empire Geo-Services, Inc. should also be retained to review final plans and specifications, and to monitor the earthwork and foundation construction, to verify that the recommendations were properly interpreted and implemented.

Additional information regarding the use and interpretation of this report is presented in Appendix F.

Sincerely,

EMPIRE GEO-SERVICES, INC.

John J. Danzer, P.E.
Senior Geotechnical Engineer