GEOTECHNICAL REPORT

For: Kingstonian Development
19 Front Street
Kingston, New York

File No. 6439

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INTRODUCTION:

The preliminary subsurface investigation for the proposed Kingstonian Development to be located at 19 North Front Street, Kingston, New York, has been completed. Six (6) soil borings have been completed by Northeast Specialized Drilling, Inc., of Liverpool, New York. Four (4) cone probes were done by Conetec, Inc., of West Berlin, New Jersey. At the one of these locations additional in-situ testing was done including shear wave velocity measurements, and Marchetti dilatometer tests. Pore pressure dissipation tests were performed at several locations to determine the depth of the groundwater table and estimate the drainage properties of the site soils. The logs of these borings, cone probe tests, and other in-situ tests, along with a location diagram, have been included in the appendix of this report.

It is my understanding that the proposed construction will include two buildings; with a bridge connecting the buildings. There may be a swimming pool. There will be up to four levels of parking garage with a series of shops and apartments resting on the top of the parking garage. I have included renderings of the planned construction in the appendix.

A site plan showing the locations of the planned building footprints, the existing structures, and the existing topography has also been included in the appendix.

The buildings will impose light to moderate loads on the foundation soils. I have included a preliminary foundation plan and table showing the estimated foundation loads in the appendix. The column loadings might range up to about 950 kips with strip loadings up to 40 kips/foot.

The scope of my services has been limited to coordinating the field and laboratory investigation, analyzing the soils information, and providing a preliminary geotechnical report with preliminary foundation design recommendations.

A primary focus of this preliminary investigation has been to determine if the proposed construction can be supported on shallow foundations supplemented by ground improvement such as dynamic compaction and possibly rammed aggregate piers. If rammed aggregate piers are needed as an extension of the dynamic compaction process the purposes might be to lower seismic shocks on buildings close to the work area or to reduce settlements in local areas.

Environmental, site design, and structural design aspects of the project should be performed by qualified others.

A supplementary subsurface investigation is planned prior to finalizing the project plans and specifications. An outline of what this supplementary investigation might include has been provided later in this report.

FIELD INVESTIGATION PROCEDURES:

The soil borings were performed with a truck-mounted drilling rig. The borings were advanced using 3.25 inch, I.D., hollow-stem augers.

Samples were obtained from the boring holes by means of the split-spoon sampling procedure. The standard penetration values obtained from this procedure have been indicated on the logs as “N” values and as blows for each six inches of penetration. Hand penetrometer readings for estimated unconfined compressive strengths were taken on some split spoon samples showing some cohesion. The results in tsf are shown on the logs.

Soil samples obtained from these procedures were examined in the field, sealed in containers, and shipped to the laboratory for further examination and classification.

A tube sample of the silts and clays was taken.

In addition to the field boring investigation, the soils engineer visited the site to observe the surface conditions.

The cone probe testing was done using a cone testing truck. A description of this vehicle and all the testing equipment used is included in the Conetec, Inc., project report in the appendix.

LABORATORY INVESTIGATION:

The samples were examined in the laboratory by the geotechnical engineer.

Nine (9) representative samples were subjected to grain size analysis testing by sieve analysis.

Five (5) samples were tested for atterberg limits to estimate the plasticity of the soils as it affects performance under load; handling properties; and drainage properties.

Forty eight (48) samples were tested for natural moisture content. Natural moisture contents allow extrapolation of grain size and plasticity information to other samples as well as providing economical information on compressibility, grain size, and handling characteristics.

One unit weight test was done on the tube sample.

The laboratory result sheets for these tests are included in the appendix.

IN-SITU TESTING:

The in-situ testing including measurement of the cone tip resistance, friction sleeve measurements, pore water measurements, shear wave velocity measurements, dilatometer testing, and pore water dissipation testing done by Conetec provide a great deal of subsurface information that supplements the standard penetration testing done with the boring rig and the laboratory testing.

The most commonly used information from the cone work is shown on the cone logs and on data sheets from the shear wave velocity testing and dilatometer testing in the appendix.

I have made reference in this report to other information coming from the basic cone values such as over-consolidation ratios, liquefaction safety factor estimates, and elastic modulus estimates that are available to me on the Conetec Data Services web site or in the Conetec "CPT and DMT Report" in the appendix of this report. If anyone would like to receive other cone data from the work done during this investigation, it can be provided.

SITE CONDITIONS:

The site is a gently sloping site with elevations of about 183 at North Front Street at the south end of the site and about 155 at the north end near Schwenk Drive. There is a low retaining wall separating east and west **parking areas** in the larger building area. The grade change is about 3 to 6 feet with the western side at a lower elevation.

The site has been in use primarily as a parking lot.

The surface appears to be generally well drained and stable.

The geology of the area features a relatively deep lacustrine soil deposit laid down in a glacial lake dating to the last post glacial epoch. The overburden soils consisting of sands, silts, and silty clays commonly extends to depths of over 100 feet. These layered soils rest on a stratum of dense glacial till which in turn rests on limestone or shale bedrocks.

These overburden soils are commonly slightly over-consolidated due to the removal of some glacial material by erosion as the Hudson River reached its present day bed or stream bottom level with elevations across the Valley becoming lower in this process.

There are existing buildings along the west side of the site and at the south end of the site which will need to be protected from construction vibrations.

SUBSURFACE CONDITIONS:

The specific subsurface conditions encountered at each boring location are indicated on the individual soil boring logs. However, to aid in the evaluation of this data, I have prepared a generalized description of the soil conditions based on the boring data.

The descriptions on the soil boring logs are those of the drill foremen. I reviewed them when I visually classified the soil samples and found the field descriptions to be good descriptions of the samples and to be matching the cone derived classifications (on the cone logs) quite well.

The cone probe logs describe a similar stratigraphy as do the soil boring logs with considerably more detail. The cone tip resistances, and friction sleeve values shown in the colored, graphical, logs provide a good estimate of the stiffness of the soil strata and the color coding provides a good general description of the soil types. In addition the cone probe logs reveal detail on the layering of the lacustrine soils with very many layers of sediments. The water levels are also shown on the cone logs.

There is a layer of in-situ fill on the site. The fill is generally loose or lower medium dense in compactness and consists of mixed soil textures with small amounts of miscellaneous fill materials such as brick fragments, cinders and ashes.. The depth of this fill varies from as much as 23 feet at the south end of the smaller building east of Fair Street to as little as 5 feet in the northern end of the larger building west of Fair Street.

Boring TB-6 describes the soils in the slope near North Front Street. The soils from elevation about 183 to about 21 feet deep or elevation 163 are layers of sand fill with traces of miscellaneous materials.

Another general pattern can be noted in the stratification of the site soils. Beneath the fill soils there is an upper series of layers of silt, silty sand, and silty clay or clayey silt soils in a loose to lower medium dense condition extending to a depth of about 40 feet. Just above a depth of about 40 feet there is a consistent pattern of sand layers which are shown in yellow on the cone testing logs.

The deeper soils below about 40 feet all the way to the dense glacial till at 94 feet to 101 feet deep the soils are uniformly layers of silts, fine sandy silts, clayey silts, and occasional thin silty clay layers in a very loose condition.

It is worth pointing out that here is a distinct difference between the subsurface conditions at shallower depths in the eastern (smaller) building and those in the western (larger) building. Some of the upper silt and silty clay strata are looser or softer than in the western building.

I have assumed for the purposes of this preliminary report that the deeper conditions are similar in both buildings. I have used the information from the boring TB-1 and cone probe at SCPT-17-01 to extend the information from the borings and probes in the eastern building for purposes of settlement estimation.

GROUNDWATER CONDITIONS:

The specific groundwater conditions are shown on the soil boring logs and on the cone probe logs. The levels on the cone probes are derived from the pore water dissipation testing at the cone probe locations. However, in general, the groundwater levels shown on the cone logs are about 20 feet below the ground surface.

The depths to ground water varied most noticeably at two locations. The samples became wet to saturated at a depth of about 15 to 17 feet at the TB-5 location and at about 22 to 25 feet at the TB-6 location. No cone probes were done near these locations to verify the saturation of soils below these depths.

Some fluctuation in groundwater levels and perched water conditions should be anticipated with variations in the seasonal rainfall and surface runoff.

ANALYSIS AND RECOMMENDATIONS:

Site Work:

The existing buildings and their foundations, surface topsoils, pavements, trees, stumps and debris should be removed prior to any ground improvement or other construction.

I recommend that dynamic compaction be used as a ground improvement method within the proposed building footprints extending at least 10 feet outside the building lines. In cases where the required ground improvement is close to an existing structure rammed aggregate piers might be used in lieu of dynamic compaction itself to control seismic velocities. Also local evaluations can be made including supplementary investigation to determine the best way to support the proposed structure in those areas.

The areas to be dynamically compacted should be graded to allow free movement of the cable crane used in the process.

The dynamic compaction process requires that the subgrade be at least 6 feet above the groundwater table. The preliminary site grading should take this into account. In local areas where groundwater might be mounded due to rain or runoff some material might need to be temporarily added to meet this requirement.

All building footprints will be compacted using the same grid patterns and applied energy.

Dynamic Compaction Recommendations:

The dynamic compaction and other ground improvement work should be done by a very experienced design/build contractor. I have provided my recommendations for this work below. The final "Work Plan" will need to be worked out by myself and the design/build contractor.

I recommend the application of a uniform amount of compaction energy over the building footprints extending to 10 feet outside the building lines. In addition to that I recommend that the individual spread footing foundation locations be treated with a pattern of drop points that depends on the size of the foundation. The larger footings may require up to five drop points each and the rest of the footing subgrades up to four drop points. The drop points would be arranged at the corners or at the corners and middle of the footing area.

I recommend that the building pads be compacted using an 8 ton weight and a drop height varying from 25 feet to 45 feet as required to compact the existing fill soils and the loose to medium dense virgin soils within the effective depth of the process. The appropriate amount of energy to be applied at each drop point can be evaluated by the response of the subgrade to the impact of the weight including the crater depth. The effective depth of dynamic compaction is generally taken to be a function of the energy of each blow in terms of height of drop and the magnitude of the weight itself and a coefficient which can vary depending on the subgrade soils, groundwater, presence of hard or soft layers, the amount of energy applied and other factors. On this site I recommend that the desired compaction can be achieved using a grid spacing of 9 feet by 9 feet. The energy can be applied in either one or two overlapping phases depending on the local subgrade conditions and the response to the dropping of the weight. The number of drops per drop point required will vary from about 5 to about 10. The need will be determined by the subgrade response.

The applied energy per unit area corresponding to the 9 foot by 9 foot grid with up to 10 drops of up to 45 feet with an 8 ton weight will be up to about 11,088 KJ/m2 in metric units. Assuming that about 20 feet or 6 meters of soil will be compacted, 230 kJ/m3 in terms of energy per cubic meter will be applied.

If any of the craters should exceed a depth of 3 feet, the crater should be filled with granular fill or crushed stone and additional drops applied until the bottom of the crater is tight or hard.

An ironing pass should follow the primary dynamic compaction work to better compact the soils above the bottom of crater depths. This is done using a weight with a lower static ground pressure and a lower drop height with impact areas overlapping.

Following the application of the dynamic compaction to the building footprint the surface should be rolled at least 7 times in each direction with a 20 ton-rated, dynamic, pad foot (sheeps foot type) roller to densify the shallow soils left disturbed by the crater formations at the drop points. The rollers with truncated pyramidal shaped "feet" will project the compaction energy deeper into the subgrade than will a smooth drum roller.

***Controlled Fill*:**

Controlled, relatively clean, granular fill can be spread in lifts not exceeding 12 inches in loose thickness. These materials should be compacted to a minimum of 95 percent of the maximum ASTM Specification D 1557-91 density, modified proctor.

Materials containing significant percentages of fine-grained soils or cohesive materials should be spread in lifts not exceeding 9 inches in loose thickness and compacted to a minimum of 90 percent of the same density standard.

On-site material may be difficult to compact during wet weather or poor drying conditions.

All controlled fill should be free of organic and/or frozen material.

Free-draining controlled fill should have less than 10 percent fines passing the #200 sieve. NYS DOT subbase items, Type 2 and Type 4 meet these requirements.

Soils compacted by the dynamic compaction procedure recommended above will not require in-place density testing.

***Building Foundations*:**

I recommend that the proposed structures be supported by spread footing foundations resting on virgin soils, controlled fill which, in turn, rests on these virgin materials or on dynamically compacted subgrades. Footings can be designed for a maximum, net, allowable soil bearing pressure of 3000 psf.

A minimum footing width of 2.0 feet is recommended for load-bearing strip footings. Isolated footings should be at least 4 feet wide

Exterior footings or footings in unheated areas should have a minimum of 4.0 feet of embedment for protection from frost action. Interior footings should have a minimum embedment of 2.0 feet below finished grade to develop the bearing value of the soils.

I recommend that 3 to 4 inches of NYS DOT subbase, Type 2 or 4, be placed over the subgrade and that the subgrade be well-tamped with a dynamic compactor. This will tighten soils loosened by the excavation and provide a uniform surface for placement of the reinforcing steel and the concrete itself.

I have estimated the settlements of footings designed for 3 ksf with the loads received on the foundation plan and schedule reduced by the estimated pressure removed from the footing location due to excavation to the design lowest finished floor elevation of about 151.5 feet. I have ignored the clear benefit of the stress relief provided by removal of the soil between footings. When the design has advanced, new settlement estimates and final foundation designs can take advantage of all stress relief from excavation as well as the final load estimates.

Basement or Retaining Walls:

All below-grade walls that retain soil should have a slotted drain pipe placed around the exterior base of the wall. The drain pipe should be a minimum of 6 inches in diameter and have a slot size suitable for the filter protection required for the select granular backfill used. A 1/8 inch wide slot will work with NYS DOT subbase items. A larger slot could be evaluated for the well graded crushed stone items. The drain tile should drain to a stormwater sewer, daylight, or a sump equipped with a pump.

The wall should then be backfilled with a controlled select granular material such as NYS DOT subbase Type 2 or 4. These gradations will work as a filter medium with a 1/8 inch slot size in the drain pipe. The material should extend away from the wall a horizontal distance of one half the height of the fill being placed. The upper 1 foot of material should be a fairly impermeable material or the backfill protected by pavement to shed surface water.

If these procedures are used, a lateral soil pressure of 52 psf per foot of retained soil can be used for design of the wall. This is an at-rest lateral soil pressure and is based on a moist unit weight of 125 pcf and an angle of internal friction of 36 degrees. A coefficient of base sliding of 0.45 can also be used for design.

Any surcharge load should also be added to the above pressures. The at-rest pressure coefficient is 0.41.

It may well be economical in the final design phase to use light weight backfills against retaining walls in some locations. Uniformly graded crushed stone, light weight aggregates, or geofoam could be used to reduce lateral pressures due to both lower unit weight and greater shear strength. This may allow economies compared to relying on the building frame to take the lateral load of conventional sand and gravel backfill.

***Ground Floor Slabs*:**

Floor slabs can be designed to rest on virgin, inorganic, soils; dymamically compacted subgrade soils; or on controlled fills resting on these materials. An 8-inch layer of well-graded, free-draining, granular material should be placed beneath the floor slab to provide drainage, act as a capillary break, and to provide better and more uniform support. I recommend that an AASHTO Size 57 stone; a NYS DOT size 1 or 1A stone be used as a slab base to allow more drainage capacity as well as good slab support.

In any areas below grade under-slab drainage is recommended to move groundwater seepage that may exceed the capacity of the slab base into the site drainage system. If water accumulated in the slab base, it might come upward at joints and create an icing problem in seasons.

The slabs should be designed or any wheel or post loads. A design coefficient of subgrade reaction of 100 pci can be used for design on granular subgrades. Area loads up to 200 to 300 psf will not require any special design attention.

The net loads in the building areas are near zero or negative which reduces any settlement due to uniform loads over large areas.

Foundation Plan and Preliminary Foundation Load Estimates:

I have included the preliminary foundation plan with a schedule of loads in the appendix. I have also included the design engineer's estimate of the average increase in net pressure on the subgrade soils due to these estimated loads broken down by areas designated A through F. These values of net average increase in pressure on the subgrade due to proposed loads by area can be compared to the load relief on the same subgrades due to the weight of soil to be excavated to reach design subgrade elevation. An estimated value for excavated in-situ soils could be assumed at 115 pcf to make this comparison.

Depending on the final design finished floor elevation for the ground slab at level -3 the net change in load is negative or only very slightly positive at the extreme north end of the two buildings.

Estimated Settlements and Design Loads:

I have estimated settlements using the Menard pressuremeter design rules. I have estimated the modulus values for the sands using the cone tip resistance values (qt) multiplied by 1.15. I have estimated the modulus of the silts and clays above a depth of 50 feet below existing grade using the qt values multiplied by 2.5. These are commonly used multipliers to calculate the Eo or Em values for pressuremeter analysis. The multiplier for the sands is lower because if an actual pressuremeter test were done the sands would show a modulus of about half of those in clays of equal elastic modulus (such as young's modulus) because the sands to not resist tension stresses that occur in PMT testing.

Below 50 feet I have used the elastic modulus values, Eo, from the dilatometer testing that was done from that level down to the glacial till. In my opinion these values are both higher and more accurate than those calculated from the cone tip (qt) values. If pressuremeter testing were done with a Menard type (stress controlled) pressurementer, it is likely that those modulus values would be even higher than the dilatometer values, but maybe not by much.

The estimated settlements will remain within the tolerable limits assumed at one inch maximum and 3/4 inches differential for the point loads or column loads up to 950 kips and strip loads up to 40 kips per foot for the buildings on shallow foundations.

I have included estimates of settlement in the appendix for three foundations where the highest point loads and strip loads are predicted for subgrade conditions depicted by the nearest boring and cone probe logs. I have chosen locations where the least load relief due to excavation is also indicated. These locations were at columns L-2, T-12, and L-6.5. The estimates were made for loads of 950 kips at L-2; 33 kips per foot at T-12; and 40 kips per foot at L-6.5. I reduced the recommended maximum allowable soil bearing pressure of 3 ksf as stated earlier in this report by the estimated stress relief due to excavation to design subgrade at 151.5. I took no settlement reduction for the soil removed between footings which provides a margin for probable future reductions in total settlements. Little change is likely to occur in estimated differential settlements.

If some of the design loads exceed these values, changes can be made to the foundation design to accommodate higher loads. If most cases, the footings can be made larger at the same or similar design pressure to gain a reduction in settlement.

It is also an option to stiffen the dynamically compacted top layer of the subgrade beyond the modulus achieved by the dynamic compaction process outlined earlier. This can be done by installing rammed aggregate piers to a depth of 8 to 10 feet at a grid spacing required to achieve the settlement reduction required. This process replaces the subgrade in areas requiring stiffening with heavily tamped crushed stone placed in cylindrical excavations augered in a grid pattern over the treated area. The modulus of the tamped stone may range in modulus up to 1000 tsf creating a layer at least the depth of the rammed aggregate piers that has a much higher composite modulus. That reduces the settlements. Where less stiffening is required rammed aggregate piers without pre-excavation could be installed.

If higher foundation loads are to be imposed, the geotechnical engineer should review these designs and provide a revised design to accommodate them along with the estimated settlement corresponding.

The structural design should consider the framing of the upper buildings at the joints in the structure of the supporting garages to better accommodate the greater potential for differential movements at those joints.

Seismic Design:

The site is a Class D site. This is based on a calculated Vs bar value of 614 ft/sec at the SCPT 17-01 location on the north end of the site. The earthquake response accelerations for the maximum considered earthquake for Kingston, NY, are as follows: (These values correspond to the IBC 2006/2009 values taken from 2003 USGS data.)

Ss	**0.172g**	**Short Period**
S1	**0.064g**	**Long Period**

The Vs values and the spreadsheet Vs bar calculation have been included in the appendix. The Conetec report provides more detail.

Liquefaction:

The cleaner sands and gravels below the water table are the most vulnerable to liquefaction during an earthquake event. Other things equal the shallower strata are more vulnerable than deeper strata.

My approach to estimating safety factors against liquefaction utilizes shear wave velocities when they are available. I calculate the safety factors based on an approach proposed by Ronald D. Andrus and Kenneth H. Stokoe II in their contribution to an article titled "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils." The article was published in the October, 2001, "Journal of Geotechnical and Geoenvironmental Engineering" in the October, 2001, issue.

I have checked two sand layers at SCPT-1701 that appear most vulnerable at that location and found that the safety factors were close to 2.0. I used the actual sieve tests from the nearby boring TB-1 to get the percentages of fines.

The application of ground improvement with dynamic compaction will increase these safety factors withing the range of effective depth plus depending on the specifics. The dynamic compaction should liquefy the strata that are vulnerable to liquefaction down to about 25 feet below the work surface.

I have assumed an earthquake magnitude of Mw=7.0 in this analysis.

Future or Supplementary Investigation:

The number of borings and cone probes done so far is relatively small for the total area of buildings planned. The focus was to get sufficient preliminary information for a preliminary design. My main focus has been to determine that a shallow foundation approach can be used economically for this design. There have been concerns expressed by others that this site has settlement issues and liquefaction issues due to the very deep loose and soft lacustrine soils on the site.

It is my opinion that the information in hand allows us to go ahead with shallow foundations from a settlement perspective. The liquefaction safety factors checked at SCPT 17-01 indicate that liquefaction issues can be addressed within the scope of the planned dynamic compaction program.

What I recommend for a scope of the additonal investigation at this point using the preliminary building design information and available subsurface information is the following:

One to two days of Additional Cone Testing with some additional shear wave velocity measurements and Marchetti dilatometer measurements.

Two days of additonal boring work.

Additional Laboratory Testing with a similar scope to that completed.

One to two days of pressuremeter testing depending on the results achieved with the additional dilatometer testing.

The principal issues include the following:

1. We need more detailed information in the south end of the smaller building.

2. We can check the shear wave velocity in some strata to determine whether any densification is required. We are unlikely to have safety factors less than 1.5 to 1.6 where liquefaction caused settlements become more significant, but we might want to reduce the possible