DENTE
ENGINEERING

ALBANY AREA
594 Broadway
Watervliet, NY 12189
Voice 518-266-0310
Fax 518-266-9238

BUFFALO AREA
PO Box 482
Orchard Park, NY 14127
Voice 716-649-9474
Fax 716-648-3521

# GEOTECHNICAL EVALUATION FOR SARATOGA TOWN HALL BUILDING SCHUYLERVILLE, NEW YORK FILE NO. FDE-05-079

## I. INTRODUCTION

This report presents the results of a subsurface investigation and geotechnical evaluation completed for the new Saratoga Town Hall Building planned for construction in Schuylerville, New York. Our services have been provided in general accord with our proposal number PFDE-05-33 dated February 15, 2005, as authorized by the Town of Saratoga.

In general, our scope of services included:

- The field location and completion of six test borings at the project site. These borings were supplemented by five borings completed by Dente Engineering in March 2003 to investigate settlement of the existing Town Hall Building.
- Part-time observation of the test borings and site reconnaissance by a Geotechnical Engineer.
- Completion of laboratory grain size distribution and moisture content testing on representative soil samples recovered from the test borings.
- Evaluation of the field exploration and laboratory testing results and preparation of this report, which presents our geotechnical recommendations to assist in planning for design and construction of the new Town Hall Building and pavements

This report and the recommendations contained within it were developed for specific application to the site and construction planned, as we currently understand it. Corrections in our understanding, changes in the structure locations, their grades, loads, etc. should be brought to our attention so that we may evaluate their effect upon the recommendations offered in this report.

A sheet entitled *"Important Information about your Geotechnical Engineering Report"* prepared by the Association of Engineering Firms Practicing in the Geosciences is presented following the title page of this report. This sheet should never be separated from this report and be carefully reviewed as it sets the only context within which this report should be used.

This report was prepared for informational purposes and should not be considered part of the contract documents. Should the data contained in this report not be adequate for the contractor's purposes, the contractor may conduct their own investigations, tests and analyses for use in bid preparation.

## II. SITE AND PROJECT DESCRIPTION

The existing Saratoga Town Hall building is located in Schuylerville, New York on a parcel of land fronting Ferry Street and bordered to the west by the Champlain Canal. It is our understanding that the Town plans to demolish the existing building, which has experienced excessive settlement, and replace it with a new one or two story structure.

A section of the USGS Topographic Map of the Schulyerville Quadrangle is provided in Appendix A. The Map shows the site location and the surrounding land use and topography. Photographs of the project site are included in Appendix B.

It is our understanding that the walls and floor of the existing building began to settle immediately after its construction. About three years after its construction, modifications were made to the foundation walls to level the main floor. The building continued to settle as much as 12 inches differentially and its walls and floors have cracked. In May 2003, Dente Engineering completed an investigation and evaluation to determine the cause of the settlement and provide options to reduce further settlement. The investigation found the site was mantled with as much as 25 feet of fill on the south side of the building and 15 feet of fill on the north side. The evaluation concluded that the building settlements were caused by consolidation of the poorly compacted fill, compression of deeper organic rich silt and peat layers, the seasonal variations in groundwater levels, and localized leakage from utilities.

Based on the previous investigation results, it is apparent that the grades surrounding the existing Town Hall building were created through the placement of fill to level the lot with Ferry Street. The grades step down to the north, such that the two story building is set into a hillside, with the upper floor grade roughly matching Ferry Street and the lower level exiting to a parking lot at the rear of the building. North of the parking lot at the rear of the building, the site grades slope down about four to six feet in elevation to an open lawn area with a few large trees.

The location of the new building on the site has not been established, and its final siting may be influenced, in part, by the results of this geotechnical evaluation. Potentially the building could be sited in the existing upper or lower level parking areas or possibly in the low lying lawn area north of the lower level parking lot.

## III. SUBSURFACE CONDITIONS

**Method of Exploration**

Five test borings (B-1 through B-5) located immediately adjacent to the existing building were completed by Dente Engineering in March 2003 to investigate the conditions which led to settlement of the building. To supplement this information, an additional six borings (B-6 through B-11) were completed across the site in areas where the new building may be located. The approximate boring locations are shown on the Subsurface Exploration Plan in Appendix C.

The borings were performed using a standard rotary drill rig equipped with hollow stem augers. As the augers were advanced, the overburden soils were sampled and their relative density determined using split-spoon sampling techniques in general accord with ASTM D-1586 procedures. Representative portions of the soil samples recovered from the test borings were transported to our office for visual classification by a Geotechnical Engineer. Laboratory grain size distribution and moisture content testing was performed on selected soil samples from the original and supplemental borings to refine the visual classifications.

Individual Subsurface Logs were prepared for the borings based on the visual classifications and laboratory test results. Logs for all the test borings completed at the site are presented in Appendix D together with sheets which explain the terms used in their preparation. Laboratory test results from the original investigation and this supplemental study are provided in Appendix E.

**Subsurface Conditions**

The Subsurface Logs should be reviewed for a description of the conditions encountered at the specific test boring locations. It should be understood that conditions are only known at the depths and locations sampled. Conditions at other depths and locations may be different, especially where fill has been placed to create the site grades. The following is a generalized summary of the subsurface conditions based on our interpretation of the field test results.

As expected, the parking lots for the existing Town Hall building were constructed over fill materials which vary in depth from about 18 to 25 feet in the upper lot and 10 to 15 feet in the lower lot at the rear of the building. No fill was found in the lawn area north of the rear parking lot. In general, the fill was composed of loose to firm sand, silt, gravel, brick, concrete, wood, mortar, ash, cinders and other miscellaneous debris. Beneath the fill were remnants of the original topsoil layer followed by a nominal five to ten foot thick layer of very soft/loose silt and/or fine sand with occasional inclusions and thin seams of organic matter. At the rear of the site, where no fill was present, the silt and fine sand layer extended about 15 feet below grade. Below the silt and fine sand sequence were deep deposits of loose fine to medium sand which extended to shale bedrock at depths of about 37 to 55 feet below grade. In some locations a very thin layer of glacial till was present between the sand and bedrock.

Based on the relative degree of "wetness" of the recovered soil samples and measurements of water in the augers at completion of drilling, it appears that the permanent groundwater levels are generally at or near the original ground surface over which the site fills were placed. Groundwater may also be found trapped or perched in layers at various depths within the fills. The groundwater levels may be influenced to some degree by the level of water in the Champlain Canal which adjoins the site.

## IV. GEOTECHNICAL RECOMMENDATIONS

### A. GENERAL

Planning for design and construction at the project site will be significantly impacted by the presence of uncontrolled fills over a wide portion of the site and the relatively soft/loose condition of the native soils underlying the fill. In their present state, the uncontrolled fills are not suitable for support of building foundations or floor slabs as they may experience excessive long-term settlements similar to the existing building. With this in mind, it is our opinion that the building design options include:

(1) The existing fills may be removed and replaced beneath the new building to allow for the use of conventional shallow spread foundations and slab-on-grade construction. The quantity of removal and replacement can be reduced by lowering the ground floor for the new building to the minimum allowable elevation and/or locating the new building as far north as possible where the fill depths diminish.

***It should be noted that several factors must be considered when evaluating the acceptability of the removal and replacement option including; the presence of the Champlain Canal along the west side of the site, groundwater conditions, and the composition of the fill to be removed.***

Depending upon the final building siting with respect to its proximity to the Champlain Canal, it may be necessary to install temporary sheeting or other bracing system along the west side of the fill excavation to maintain the integrity of the Canal embankment and limit the entrance of groundwater into the excavation. Additional investigation, beyond the scope of this study, must be performed to evaluate the Canal construction and develop appropriate means to protect it during the excavation and replacement work. This work may need to be coordinated with and approved by the appropriate State agencies in charge of Canal operations.

The excavation of existing fill may intercept pockets or layers of groundwater, and dewatering of the excavations may therefore be required. As noted above, the groundwater levels and quantities may be influenced to some degree by the presence of the Champlain Canal along the west side of the site. It should be assumed that the excavated fills, due to the abundance of miscellaneous debris, cannot be reused beneath the new building area.

(2) If the existing fill is left in place, the building foundations <u>and</u> floor slabs should be supported on piles extending to bedrock. In this case, a drilled micro pile system, similar to that previously recommended as an option for underpinning of the existing building, should be used instead of driven piles. Driven piles are not recommended for this project as obstructions in the fill may block their penetration.

If the fill is left in place beneath the pavements adjoining the pile supported building, utilities buried within them will settle as the fill continues to consolidate. For this reason, flexible connections will be required where the utilities enter the building to account for the abrupt differential movements which may occur.

(3) The new building may be located north of the rear parking lot in areas where no fill is present to allow for the use of conventional shallow spread foundations and slab-on-grade construction. This area of the site is low lying and may require several feet of fill to raise the grades above flood levels. The native soils over which the fill will be placed are very soft/loose and it should be expected that they will consolidate under the weight of the fill. Accordingly, if this option is selected, the fills should be placed several weeks to months before building construction takes places to allow the consolidation to occur. Settlement monitoring plates should be installed at the time the fill is placed so that it can be verified when the consolidation is complete and construction can begin.

(4) Removal and replacement of the existing fill beneath new pavements would be costly and is not necessary provided that it is understood that the pavements may settle and require periodic maintenance. As previously noted, utilities buried within the fill will also settle as the fill continues to consolidate.

(5) Other options for construction of the building over the existing fill were considered, but determined to be not suitable for the site conditions. These options include deep dynamic compaction of the fill or in place treatment of the fill through the injection of grout. For deep dynamic compaction, a heavy weight is repeatedly dropped from a selected height on a grid pattern across the site. We dismissed this option because the vibrations it induces may impact on the adjoining roadways and Champlain Canal embankment, and it may not produce adequate results with the type of material (i.e., large pieces of concrete, tires and organic matter) which may be contained within the fill. The in-place treatment of the fill with grout injection is also poorly suited for the site conditions because the excessive quantities of grout may be required and the added weight of the grout on the underlying native soils may induce excessive long-term settlements. Similarly, stone columns are not considered well suited because of the possible obstructions within the fill.

Based on the test boring information and seismicity of the project area, the potential may exist for the very soft/loose silt and sand layers beneath the site to liquefy during an earthquake and cause settlement of buildings supported on them. Where the potential for liquefaction exists, the New York State Building Code requires that measures be taken in design to account for loss of soil strengths and settlements which may occur. These measures may include ground stabilization, selection of appropriate foundations such as piles, and appropriate structural systems in the building to accommodate expected settlement. We note that the information obtained from test borings is rudimentary with regards to evaluation of the liquefaction risks. To fully evaluate the liquefaction potential and its impact on building design and the recommendations contained herein, specialized cone penetrometer testing should be performed at the site after the planned building area is selected. The cost for this testing and evaluation would range from about $6,000 to $8,000.

## B. SITE PREPARATION AND EARTHWORK

The requirements for site preparation will vary depending upon the option selected for building location and support as follows:

(1) If the existing fill is left in place and the building supported on piles, site preparation will simply entail demolition and removal of the existing building, stripping of topsoil and organic matter, and leveling the planned building site for construction. Existing utilities should be removed and rerouted as required.

(2) If existing fill is to be removed to allow the use of spread foundations and slab-on-grade design, the excavation limits should extend beyond each side of the new structure a distance equal to at least one-half the depth of fill removed. The temporary excavation side slopes should be no steeper than one vertical on 1.5 horizontal. ***As previously discussed, excavation bracing may be required along the west side of the site to protect the adjoining Champlain Canal from disturbance and possible serve as a groundwater cutoff.***

Dewatering should be performed if groundwater is encountered to allow the excavation and replacement work to be completed in the dry. Dente Engineering should observe the excavations to confirm that all unsuitable materials are removed. The unsuitable materials consist of existing fill and underlying surficial organic matter.

(3) If the building is constructed at the north end of the site where no fill is present, site preparation should commence with clearing of trees and stripping of topsoil and surficial organic matter. Prior to placing fill to raise site grades, the subgrade surface should be compacted using a steel drum roller with a static weight of at least seven tons. The roller should operate in its static mode and complete at least three passes over the entire subgrade surface. Any areas which pump or weave, and do not stabilize under repeated passes of the roller should be investigated to determine the cause and stabilized accordingly. Dente Engineering should observe the compaction work and approve the final subgrades.

Prior to placing fill over the approved subgrades, settlement plates should be installed at locations to be selected based on the final location and configuration of the building.

The plates should be monitored daily as the fill is placed and weekly thereafter until it is determined that consolidation of the underlying soils has stopped.

(4) Proposed pavement areas should be stripped of existing asphalt, topsoil and surficial organic matter. The subgrades should be compacted and stabilized as detailed in Item (3) above, with the exception that the roller should operate in its vibratory mode over the existing fills which are to be left in place.

An imported Structural Fill should be used as fill and backfill beneath the proposed building areas. The imported fill should consist of well graded, sound, durable sand and gravel which conforms to the requirements for Type 4 material stipulated in Section 304 of the New York State Department of Transportation (NYSDOT) Standard Specifications for Construction and Materials. The fill should be placed in uniform loose layers no more than about one foot thick, where heavy vibratory compaction equipment is used. Smaller lifts should be used where hand operated equipment is required for compaction. In either case, it is recommended that each lift be compacted to not less than 95 percent of the maximum dry density for the soil established through the Modified Proctor Compaction Test.

## C. SEISMIC DESIGN CONSIDERATIONS

For seismic design purposes, we have evaluated the site conditions in accord with Sections 1615 and 1616 of the New York State Building Code. On this basis, the following seismic site classifications and design parameters may be assumed for preliminary planning purposes pending the further evaluation discussed below. The mapped spectral response accelerations which were used to determine the design accelerations were based on 1996 data obtained from the USGS Earthquake Hazards Program Latitude-Longitude Lookup website.

Seismic Site Class: E - Soft Soil Profile

Short Period Site Coefficient: $F_a = 2.34$

1-Second Period Site Coefficient: $F_v = 3.50$

Short Period Design Spectral Response Acceleration: $S_{DS} = 0.466$

1-Second Period Design Spectral Response Acceleration: $S_{D1} = 0.221$

Seismic Design Category: D (for Seismic Use Group I or II)

The very soft/loose silt and sand deposits found beneath the fill at the project site may be susceptible to liquefaction during an earthquake. This potential and its impacts on planning for building design and construction cannot be fully evaluated using the existing test boring information. Accordingly, as previously discussed, we recommend that specialized cone penetrometer testing and evaluation be conducted at the site to determine the liquefaction risks and impacts of planning.

## D. SPREAD FOUNDATIONS

Assuming the site is prepared as recommended in Section IV.B, the new building foundations may be proportioned using an allowable net bearing pressure equal to 3000 pounds per square foot (psf) assuming they bear upon at least three feet of Structural Fill. If the foundations bear upon less than three feet of Structural Fill, the design bearing pressure should be reduced to 1500 psf with 1.5 feet of Structural Fill beneath them, and 1000 psf should they bear directly on the indigenous site soils.

The foundations should have a minimum width of two feet, even if this results in a bearing pressure which is less than the maximum allowable. Exterior foundations should bear at least four feet beneath final adjacent exterior grades to afford frost penetration protection. Interior foundations, in heated areas, may bear two feet beneath the interior floor slabs, with the provision that the tops of all footings should be at least six inches below the bottom of the overlying floor slab.

Settlement of the building foundation will be related to the care exercised during the site and foundation bearing grade preparations. Assuming that standard care is taken during construction, and the bearing surfaces are firm and stable, we expect that total foundation settlements will be less than one inch. The settlements may occur over a period of a few days to weeks after construction is complete and each load increment is applied.

## E. MICRO-PILE FOUNDATIONS

The micro-pile design for the new building is similar to that we previously recommended for underpinning of the existing structure. The piles should have a minimum six inch diameter and they should develop their capacity through friction developed in a bedrock socket. The piles may be permanently cased or uncased as desired and reinforced as necessary. It should be assumed that the piles will at least require a temporary casing to the surface of rock to maintain stability of the holes during their construction.

The tabulation below provides a summary of allowable compressive load capacity for two pile diameters and rock socket lengths. Capacities at other diameters and socket length should not be interpolated. The uplift capacity can be calculated as 65 percent of the tabulated compressive loads to account for an increased factor of safety. Piles should be placed no closer than about 30 inches. To prevent disturbance to the setting grout, no new pile installation should be permitted within 25 feet of a previously installed pile until at least 24 hours have passed.

| **Bedrock Socket Length (feet)** | **Allowable Compressive Load Capacity (kips)** | |
|---|---|---|
| | **6" Diameter Socket** | **8" Diameter Socket** |
| 5 | 75 | 105 |
| 10 | 130 | 180 |

Piles will settle upon loading, primarily through elastic shortening of the piles. We expect that total and differential movements at the top of the pile will be less than one-half inch. The movements should occur quickly as the loads are applied.

## F. RETAINING WALLS

Any building or site walls that retain earth should be designed to resist lateral earth pressures together with any applicable surcharge loads. If the walls are free to deflect as the backfill is placed or surcharge loads applied, "Active" earth pressures may be assumed. If the walls are braced prior to backfilling or applying surcharge loads, "At-Rest" conditions should be assumed. The following design parameters are provided to assist in determining the lateral wall loads, whichever apply:

- Coefficient of "At-Rest" Lateral Earth Pressure $K_o = 0.50$
- Coefficient of "Active" Lateral Earth Pressure $K_a = 0.33$
- Coefficient of "Passive" Earth Pressure $K_P = 3.0$
- Total Unit Weight of Soil and Compacted Backfill $Y_T = 120$ pcf

Assuming that the Seismic Design Category for the project is "D", the lateral pressures induced on the walls during an earthquake must be included in design. The dynamic load component of the wall pressures should be assumed to act at approximately 0.6H above the base of the wall, where H is the height of the wall. The dynamic load can be estimated at approximately $7.2 \times H^2$ in pounds per square foot. If the recommended cone penetrometer testing and evaluation is performed at the site, it may be possible to change the Design Category to "C", and in this case the earthquake induced forces on the wall can be ignored.

The design parameters assume that the walls are backfilled with imported Structural Fill and that a foundation drain is installed to prevent water from becoming trapped in the backfill soils and creating hydrostatic pressures on the wall. The drain should be a minimum four inch diameter perforated PVC pipe embedded in at least 12 inches of clean one and two inch size crushed stone. The stone should be wrapped in a filter fabric (Mirafi 140N or equivalent) to inhibit siltation.

## G. SLAB-ON-GRADE

The building floor slabs may be designed in accord with the recommended procedures of the American Concrete Institute or Portland Cement Association. The floor slabs should be cast upon a vapor barrier placed over a nominal four inch thick stone layer composed of ASTM C-33 Blend 57 material. At least 12 inches of Structural Fill should be provided beneath the stone layer. The slabs may be designed using a Modulus of Subgrade Reaction equal to 150 pounds per cubic inch (pci) at the top of the stone base layer.

It should be understood that frost heave may occur beneath sidewalks or pavements, and the heave may be differential, particularly where sidewalks and pavements meet building doorways. If these conditions exist and are undesirable, a 16 inch thick crushed stone base course composed of ASTM 57 Blend Stone with underdrains should be placed beneath the sidewalks or pavements to limit heave to generally tolerable magnitudes for most winters.

## H. PAVEMENTS

Assuming the site is prepared as recommended, flexible pavements may be constructed for the entrance drives and parking lots. If the pavements are subject primarily to automobile with occasional delivery truck traffic, the following pavement section may be considered for use at the project site.

| FLEXIBLE PAVEMENT SECTION<br>Saratoga Town Hall - Schuylerville, New York | | |
|---|---|---|
| **Course** | **NYSDOT Specification** | **Layer Thickness** |
| **Top:** Asphaltic Concrete | Section 403 - Type 7 | 1" |
| **Binder:** Asphaltic Concrete | Section 403 - Type 3 | 3" |
| **Base:** Processed Sand and Gravel | Section 304 - Type 4 | 12" |
| **Fabric:** Mirafi 500X or equivalent (See Note) | NA | Single Ply |

Note: The fabric is required only in areas of the site where the pavements are constructed directly over the indigenous site soils.

All materials and construction should conform with the NYSDOT Standard Specifications for Construction and Materials. The base course materials should be compacted to 95 percent of the maximum dry density for the material established through the Modified Proctor compaction test, ASTM D1557. Prior to placing the base, the subgrade should be shaped to promote runoff to the pavement edges and sealed with a steel drum roller. The base course materials for the pavements should be day-lighted to drainage swales or connected with edge drains.

## I. SUPPLEMENTAL STUDY

As previously discussed, cone penetrometer testing should be conducted at the site after the building location is finalized to serve as the basis for a liquefaction analysis. At least two test holes should be advanced to refusal, with shear wave velocity testing conducted at five foot intervals in each location.

Additional investigation and analysis must also be performed, if the option to remove and replace existing fill is selected, to evaluate stability of excavations and the need for bracing along side the Champlain Canal. The scope of this additional work should be developed based on the final siting of the new building.

## J. CONSTRUCTION MONITORING

Dente Engineering should observe removal and replacement of existing fill to document that all unsuitable materials are removed, observe and approve compaction and stabilization of floor slab and pavement subgrades, and document installation of piles should they be selected for use. If the building is located at the north end of the site where fills need to be placed to raise site grades, Dente Engineering should review settlement plate monitoring results to confirm when consolidation of the subgrade soils is complete and construction may commence. It should be noted that the actual subsurface conditions will only be known when excavated, and the presence of the Geotechnical Engineer during construction will serve to validate the conditions assumed to exist and design recommended in this report.

## V. CLOSURE

This report was prepared for specific application to the project site and construction planned. It was prepared on the basis of a limited number of explorations at discrete locations on the site. As previously discussed, Dente Engineering should be retained during construction to validate the actual site conditions are similar to those assumed for development of the recommendations contained in this report. Dente Engineering should also review plans and specifications related to foundations, pavements and earthwork prior to their release for bidding to confirm that the recommendations were properly implemented.

This report was prepared using methods and practices common to Geotechnical Engineering, no other warranties expressed or implied are made.

We appreciate the opportunity to be of service. Should questions arise or if we may be of any other service, please contact us at your convenience.

Yours Truly,
Dente Engineering, P.C.

Edward C. Gravelle

Edward C. Gravelle, P.E.
Senior Geotechnical Engineer

Fred A. Dente, P.E.
President