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# Building Design and Construction over Organic Soil

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# Building Design and Construction over Organic Soil

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SYNOPSIS: A lowrise office building was constructed on a mat foundation over a thick peat deposit that had been preconsolidated beneath surface fill. Environmental restrictions prevented use of deep foundations for fear that penetration through an aquaclude would permit contamination of a deeper water table. This paper describes the laboratory testing and field instrumentation programs, as well as the special geotechnical and structural analysis undertaken for the design and construction of this project. Included in the program were long-term consolidation tests, pressuremeter tests, use of heave markers, inclinometers and pore pressure piezometers. A site history was also developed to define the extent and nature of the surficial fill. To achieve much of the anticipated initial settlement, the basement was temporarily flooded, thus preloading with the full building weight. Water was removed as construction proceeded so that the full building weight was always maintained. Actual settlement was observed to agree fairly well with predicted settlements.

## INTRODUCTION

Construction over organic soils has historically been a problem due to the typically low strength and high compressibility that is common to these materials. Designs usually call for supporting the total structure, including floor slabs, on deep foundations extending through the compressible deposits. However, this often leads to difficulties with entering utilities and any attached structures. Alternatively, if the structure is floated over the organic soils in an effort to try to have everything settle together, there is the difficulty of predicting the magnitude and rate of the anticipated settlement.

In the winter of 1984, STS Consultants, Ltd. undertook a project in conjunction with KKBNA, Inc. and Lohan Associates to act as the geotechnical consultant for a proposed office building to be constructed in upstate New York. The building location was to be located over a peat bog buried beneath old surface fill. The design height was to be four stories with a basement, covering approximately 32,500 square feet with column base sizes of 30 feet by 30 feet. The estimated maximum interior column loads ranged from 800 to 1000 kips and the exterior maximum column loads ranged from 550 to 700 kips.

Because of pollutants in the surface fill and the fear of groundwater contamination if deep foundations were used which would puncture the clay aquaclude underlying the organic deposit, only shallow foundation solutions could be considered.

## SITE HISTORY

A history of the project area was developed indicating that the site was, at one time, occupied by a chrome tannery plant with operations dating back to the early 1900's. According to early drawings of the site, it appeared that the majority of the area was occupied by hide houses, tannery buildings, small storage sheds and above grade storage tanks. The hide houses were constructed in 1910 and many of the storage buildings were constructed in the 1930's. At the time of the soil boring exploration program for the current structure, no buildings were standing on the site. However, there were remnants of floor slabs and foundation walls throughout. It was discovered from discussions with local people that the buildings were supported on relatively shallow spread footing foundations and that no deep foundation systems had been used on the site. It was also determined that the practice existed at one time of covering the tannery waste in the storage pits with clay fill to minimize odors.

## SUBSURFACE CONDITIONS

Analysis of the geologic activity in the region indicated a thick stratum of outwash sand and gravel deposited by glacial stream channels overlying till and bedrock at a depth of about 65 feet. Following deposition of the outwash, it appeared that a channel flowing roughly in a north-south direction was eroded in the outwash by a tributary of a nearby river. This channel appears to have been naturally dammed giving rise to a quiet depositional environment in which clay and silt were laid down. The clay is discontinuous and appears to have been breached by a rejuvenated stream which resulted in deposition of coarse grained alluvial sand and silt. In more recent geologic time, organic soil accumulated in a swampy environment associated with the channel. Aside from this, shallow and deep deposits of fill, associated with the old tannery, were found at various locations.

Fortunately, although organic deposits and peat bogs can vary widely in their physical and chemical properties, depending upon the percent of organic matter and water content, at this site the organic deposit was generally fine grained and contained significant percentages of nonorganic solids. The thickness of the deposit ranged from 0 to 25 feet in the building location and typical water contents were on the order of 100 percent. Due to the age of the tannery plant, it was assumed that the organic deposit had been consolidating under the weight of 10 to 15 feet of fill that had been in place for approximately 80 years. Therefore, the scope of this study was limited to the less compressible end of the organic deposit range and also to a deposit that had been subject to significant preconsolidation. A typical soil profile through the building site is shown on Figure 1 and the range of thickness of peat below the foundation is shown in Figure 2.

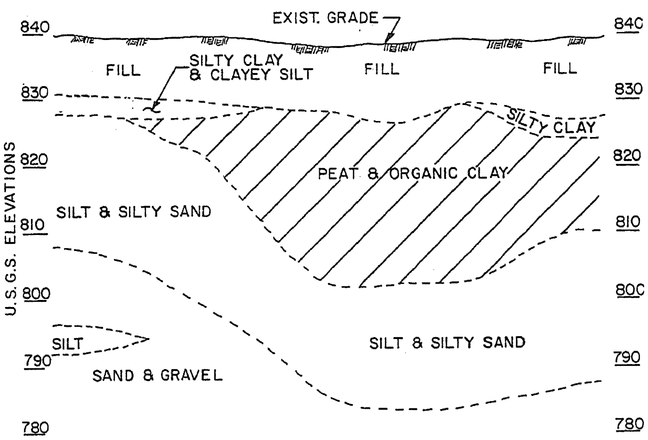


Fig. 1 Typical Soil Profile

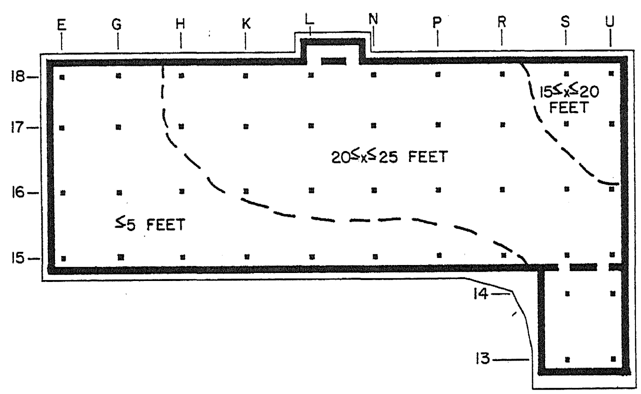


Fig. 2 Peat Thickness Below the Foundation

## LABORATORY AND FIELD TESTING PROGRAM

The testing program consisted of performing water content, density, unconfined compressive strength, organic content, Atterberg Limits, consolidation tests and grain size distribution tests on representative soil samples. In addition, in-situ pressuremeter tests were performed to obtain information on the deformation properties of the organic soils in-situ. Average values for the peat are shown on Table 1.

Table 1
Average Values for the Peat

Water Content: 92% (range 64%-146%, 26 samples)
Dry Density: 50 pcf (range 19 pcf-89 pcf, 23 samples)
Unconfined Compressive Strength: 0.54 tsf (range 0.1 tsf-1.0 tsf, 23 samples)
Organic Content: 6.2% (range 2.6%-13.1%, 23 samples)
Atterberg Limits - LL: 73 (range 44-112, 6 samples)
PI: 18 (range 14-24, 5 samples)
Grain Size Distribution
.002mm : 15% passing (range 10%-21%, 5 samples)
.005mm : 23% passing (range 16%-34%, 5 samples)
#200 sieve : 80% passing (range 72%-99%, 5 samples)
#10 sieve : 98% passing (range 94%-100%, 5 samples)
#4 sieve : 100% passing (range 99%-100%, 5 samples)
Consolidation Test Results-Standard Method
$P_c$ : 0.81 tsf (range 0.67 tsf-0.95 tsf, 2 samples)
$C_c$ : 1.8 (range 0.8-3.57, 2 samples)
$C_r$ : 0.15 (range 0.1-0.2, 2 samples)
Pressuremeter Test Results
$P_f$ : 1.5 tsf (range 1.3 tsf-1.7 tsf, 3 tests)
$P_1$ : 2.8 tsf (range 2.5 tsf-3.0 tsf, 3 tests)
$E_d$ : 25 tsf (range 21 tsf-32 tsf, 3 tests)
E+: 36 tsf (range 31 tsf-45 tsf, 3 tests)

Since anticipated settlement of the organic soils was of primary interest, two types of consolidation tests were performed. The standard consolidation test using a load increment ratio of one and a 24 hour loading sequence was performed, as well as second type of test in which the anticipated design loading condition was placed directly on the sample and left there for the duration of the test.

Initial analysis of the standard consolidation test data did not indicate anything unusual. The deposit appeared to be fully consolidated under the current overburden. The estimated preconsolidation pressure was approximately equal to the calculated overburden pressure. In addition, the time settlement curves showed fairly normal S or C type curves.

In the more unconventional single load long-term test, the results showed signs of A steepening in the slope of the secondary compression curve at about 10,000 minutes. The results of this test are shown in Figure 3.

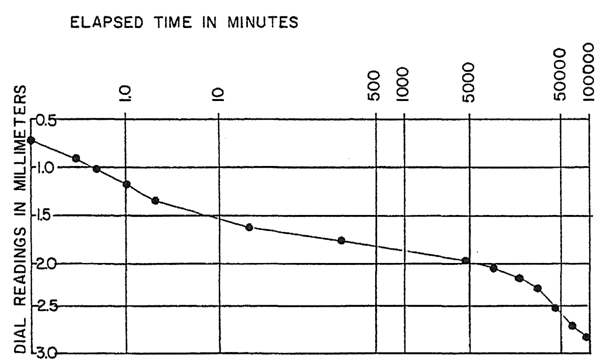


Fig. 3 Single Load Consolidation Test

## STRUCTURAL DESIGN CONSIDERATIONS

The project consisted of two L-shaped building facing each other to embrace a well landscaped court, with a bridge link at each end of the building. Because of site limitations, one building (Building A) was located on natural, medium dense granular soils, while the other (Building B) was resting on the deep organic deposit.

Structurally, the floor was concrete fill on composite metal deck supported by composite beams and steel columns. The lateral loads were resisted by the frames. The decision to use this system was based on a cost comparison of many concrete and steel floor framing schemes.

During the foundation analysis, the deep foundation alternative was eliminated to avoid penetration through the underlying aquaclude. In addition, the construction schedule could not allow for removing and replacing over 35, 000 cubic yards of organic material. Therefore, the mat foundation became the apparent solution for Building B and conventional spread footings were used for Building A.

Two major structural problems had to be resolved:

1. How to maintain a stable floor elevation, especially at bridge links between the two buildings, and

2. How to eliminate or minimize the stress induced to the steel moment frames above by the foundation movement (or differential movement).

A mat foundation on top of the organic soil would cause large initial and continual long term (over 50 years) settlements. These large settlements are difficult to predict with the accuracy desired by the architect and structural engineer. To solve these problems, the following steps were undertaken:

A. To maintain a stable building elevation after construction:

   A full basement was proposed under Building B, even though the project required only a partial basement for mechanical equipment. The basement was designed to be water tight and was utilized as a floating foundation. It was placed at a calculated depth to balance the weight of the excavated soil and the total weight of the proposed building and its foundation, plus 100% of the superimposed dead loads and 10% of the live loads (the design live load was 80 psf). The anticipated initial settlement due to future transient live load was acceptable.

B. To control the elevation of Building B during construction and to minimize the effects of heave from excavation of the organic deposit and the time required for reconsolidation:

   Water was poured into the basement as soon as the basement slab and walls were completed. The amount of water was equal to the weight of the superstructure plus 10% of the live load. It was intended to start the reconsolidation as early as possible so that it could reach a stable condition by the time all floors were installed. Immediately after each floor was completed, the same amount of water by weight was pumped out from the basement to avoid any over consolidation.

   Because the organic deposit was not at a uniform depth beneath the building, the reconsolidation was not uniform. In fact, a maximum of 1-3/4" differential settlement was estimated. In order to maintain all floors level, extra long anchor bolts were installed under each steel column with double nuts for adjustments. Adjustments were not made until just before the moment frames were connected together.

C. To avoid or to minimize initial stress in the moment frames caused by the differential settlements:

   During the steel erection, all beams to column connections were partially tightened initially. Surveys were made monthly to determine the rate of increase in foundation settlement. Final column height adjustments and tightening of all loose bolts were made after the settlements were considered to be relatively stable.

## SETTLEMENT ANALYSIS AND PREDICTION

Since the design concept was based on setting grades such that the weight of the excavated soil was equal to or greater than the total weight of the proposed building including the weight of the planned mat foundation, it was planned that settlements would be limited to reconsolidation of any rebound occurring during excavation for the mat foundation plus continuing long-term secondary compression. Using both the consolidation test data and the pressuremeter test data, calculations were made for the predicted rebound and resettlement under building load. These calculations indicated rebound and resettlement of just over 4 inches.

Based upon past experience of rebound being less than predicted, a heave and resettlement of 3 inches was predicted where the organic deposit was greater than 20 feet, with an additional long-term settlement of 3 inches over the next 50 years as a result of continuing secondary compression. Initially, this 3 inch settlement prediction was based on extrapolation of the straight line portion of the secondary compression line on the long-term load test. However, as the long-term load test results continued to be obtained, a disconcerting trend of ever steepening slope on the secondary compression line was noted. Extension of this slope would result in substantially increased settlement at 50 years. However, if it were assumed that the real current starting point should be the 80 year line so that the true settlement of interest would be the time frame of 80 years to 130 years, the predicted 50 year settlement is only 3 inches. Thus, the 50 year prediction for secondary compression was maintained at a maximum of 3 inches.

## CONSTRUCTION INSTRUMENTATION AND PERFORMANCE

In order to monitor building performance, 7 heave markers were installed prior to excavation. These consisted of concrete plugs with slightly protruding steel bars placed in large diameter boreholes to a level just below the planned mat excavation level. The boreholes were backfilled with a bright dye to facilitate finding the heavy markers after excavation. Despite precautions, some of the heave markers were destroyed during excavation.

Inclinometers were also installed just outside the perimeter of the planned excavation to monitor any lateral movements. Unfortunately, the contractor elected to use much steeper excavation slopes than recommended and a classic slope failure occurred destroying the inclinometers, indications of which can be seen in Figure 4 and 5.

After excavation and prior to mat construction, several pore pressure measuring units were installed so that pore pressure build up could be observed and pore dissipation monitored during building construction. Typical results as compared to the vertical movements measured by the heave markers are shown on Figure 6.

Surveys of elevations at the top of all concrete piers were made at the time when the basement was completed; at the time the basement was filled with water, and monthly afterward. The building elevation reached reasonably stable conditions after the third floor steel and metal deck were installed. Column adjustments and final tightening of connections were made at that time.

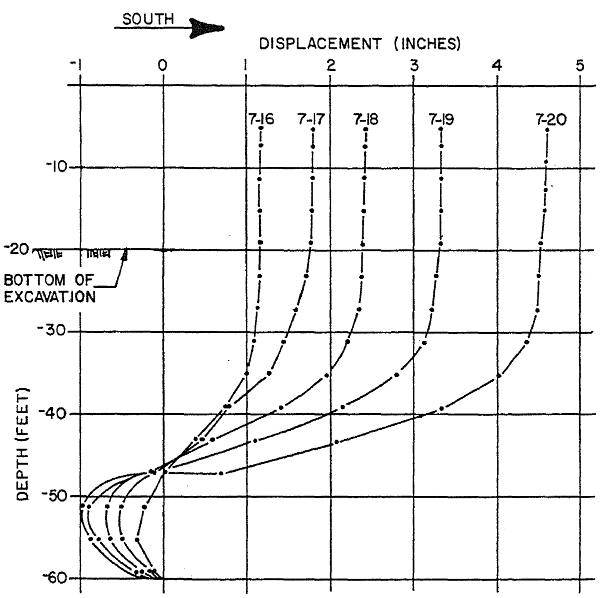


Fig. 4 Inclinometer Movement, North Side

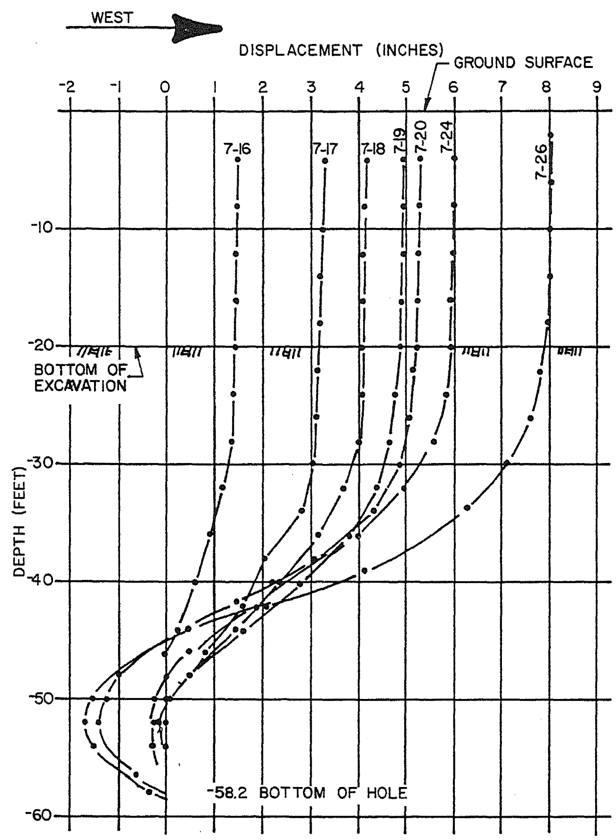


Fig. 5 Inclinometer Movement, East Side

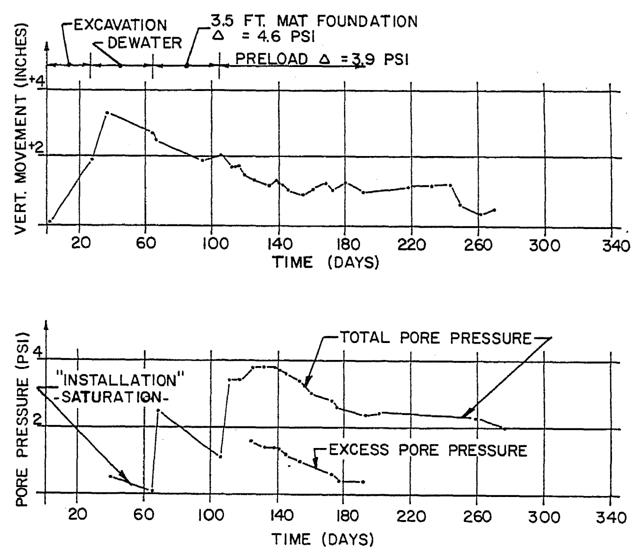


Fig. 6 Typical Heave Marker and Pore Pressure Results

## COMPARISON OF PREDICTION AND PERFORMANCE

The comparison of prediction and performance up to 5 months following building completion was remarkably close considering the nature of organic soil. Representative heave markers heaved approximately 3 inches and reconsolidation under a full building load (actual average load of about 1200 pounds per square foot) was also in the 2 to 3 inch range where the organic deposit was thickest.

There were two exceptions to this general performance. The first occurred at the northeast corner of the building where a major slide had taken place during excavation. At this corner there was a sharp increase in settlement during the water loading stage to a point where it appeared that active bearing capacity failure was occurring. In order to decrease the load at that corner, it was necessary to excavate a portion of the backfill placed against the basement walls and replace the heavy backfill with light weight fill. When this operation was completed, the rapid increase in the rate of settlement in the corner decreased to the same rate as the rest of the mat over comparable organic soil thickness. It appeared that the slope failure had so weakened the organic soil as to create the observed rapid settlement phenomenon.

The other anomaly was at the southeast corner of the building where the observed settlement was moderately greater than predicted but where there was not the excess of backfill surcharge. However, a major sump pit and pump had been located at this corner and it is believed that dewatering occurring at this corner may have created additional consolidation effects.

Predicting how long primary consolidation should take was difficult because of significant variations in the laboratory results. The time for 100 percent primary consolidation varied from less than 1 minute to more than 10 minutes. Thus, calculations of 80 percent primary consolidation occurred in time frames ranging from weeks to months. A best guess was made during construction that 80 percent pore pressure dissipation occurred within a two month period. Interpretation of the data was difficult because dewatering was going on during initial mat construction and initial pore pressure observation readings. At first, pumping was going on out of two separate pits. Once the mat and building walls were constructed, pumping on the north side of the building stopped but continued for a while on the south side. Thus, there was a tendency for the water table to build up on the north side of the building and the pore pressure readings rose independent of any building load increase.

During the months following construction, estimates of the actual secondary compression settlement were difficult since 100 percent primary consolidation had not fully occurred and the magnitude of settlement over that time had been within the up and down accuracy of the surveyors. However, it would appear that secondary compression was occurring at a rate no greater than predicted.

## CONCLUSIONS

From the data presented herein, it can be concluded that heave and resettlement can reasonably, but conservatively, be predicted in organic soils using consolidation or pressuremeter theory as long as the preconsolidation pressure in the soil is not exceeded. Furthermore, the secondary compression in organic soil does not appear to plot as a consistent straight line on a semi-log plot but rather as an initial straight line followed by a suddenly increasing slope after 2 to 4 weeks followed by a decrease again after an extended period of 6 months to a year on a laboratory 1 inch sample. Finally, the importance of long-term measurements in the laboratory and in the field must be stressed as it is necessary to determine if there are universal relationships that can be applied to the specific organic soils at a given project site.

## ACKNOWLEDGMENTS

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