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SYNOPSIS: The design, construction, and performance of several building foundations and temporary earth retaining structures located in the downtown area of White Plains, New York are presented in this paper. High rise structures were supported on shallow mat or spread foundations bearing on erratic saturated alluvial silt and sand deposits. Additionally, the construction of two and three level underground parking structures required the use of cantilevered and braced excavation support systems to retain the adjacent streets and utilities. Several assumptions were required to design and predict the performance of the building foundations and retaining structures. The accuracy of these assumptions was verified through the use of precise field measurements during and after construction. The results of these field measurements and comparison with predicted values are presented and discussed.

INTRODUCTION

Foundation problems had impacted the growth of the White Plains core area since the founding of the City. Located in prestigious Westchester County, just 12 miles north of New York City, the City of White Plains had experienced prosperity in certain areas while others were depressed and economically unproductive. While the east side flourished and major structures were constructed on competent bearing materials, the west side remained under-utilized and was occupied by substandard small buildings.

During the 1970's, the structures in the western portion were totally demolished during the early stages of the urban renewal program but only the surface problems were cleared; the complex subsurface soil strata remained to be dealt with by future redevelopment. The difficult subsurface conditions and asssociated high cost of foundations continued to hamper the redevelopment effort and the land remained vacant for many years.

Market forces demanded high rise, high quality structures and underground parking structures were required to satisfy zoning ordinances. The subsurface soil conditions with erratic layers of sensitive "bull's liver" silt, pockets of loose and variable density sands, a deep bedrock stratum, and a shallow groundwater table unfavorably impacted this type of construction and created numerous design challenges.

The authors became involved with the first building of the reconstruction effort in 1974 and subsequently for an additional 20 structures within the White Plains core area. The extreme subsurface variations coupled with the fact that conventional soil sampling was unreliable, complicated the design of cost effective foundation and excavation support systems. Since numerous design assumptions were required, it was necessary to confirm these during construction through precise monitoring under actual fully loaded conditions.

As performance results became available, more confidence in various design procedures resulted, and it was possible to perform refinements or "fine tune" designs to achieve additional efficiency and related savings in construction costs for shallow foundations and support systems for excavations. A series of case histories are presented which illustrate the design and analysis procedures utilized on some of the projects. Performance results are provided for these projects as well as other projects not specifically discussed in detail. A site location map showing the project areas to be discussed is presented in Figure 1.

SUBSURFACE CONDITIONS

The downtown area is generally underlain by fill material, river alluvium, glacial till and gneiss bedrock. The fill consists of building materials mixed with soils and has been placed within the past 200 years. The river alluvium consists of sand and discontinuous silt deposits and is of the Holocene or the late Pleistocene (glacial) epoch. The glacial till of the Pleistocene epoch is composed of a heterogenous mixture of silt, sand and gravel soil with occasional boulders. The Fordham gneiss formation of the Precambrian period is predominantly granitic with occasional schistose and quartzose zones.

The stratigraphy beneath the Westchester Financial Center and the Gateway Project sites is consistent with the general subsurface conditions presented above with the exception of the absence of a continuous alluvial silt deposit beneath the Gateway sites. The general subsurface conditions beneath the downtown area and the location of the subject buildings are presented in Figure 2.

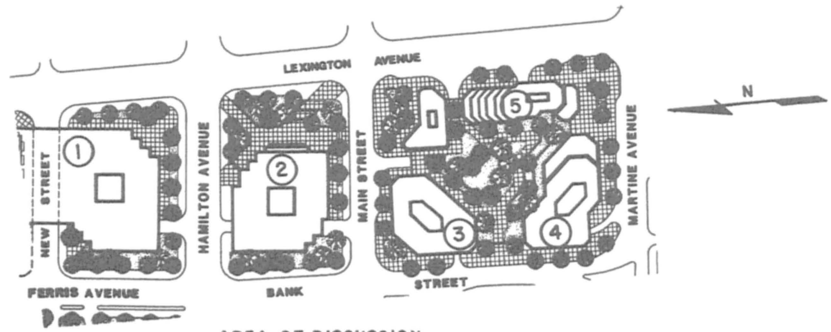


FIG. 1 - SITE LOCATION MAP

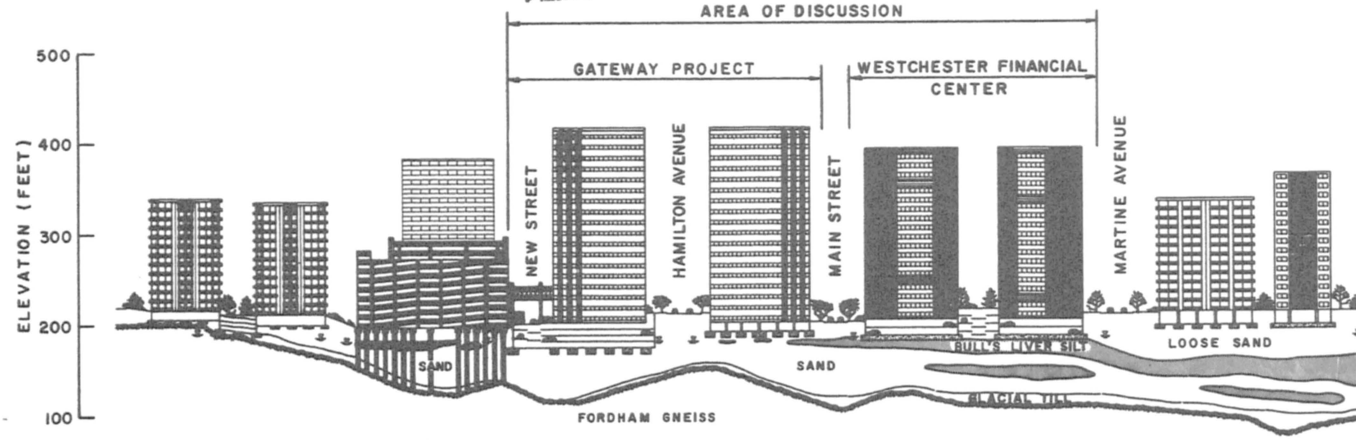


FIG. 2 - GENERAL SUBSURFACE PROFILE

The Westchester Financial Center is underlain by saturated alluvial silt and sand deposits. The silt deposit exhibits extreme dilative characteristics and is locally known as "bull's liver" due to its shiny appearance. The silt is generally encountered at or below the groundwater level and possesses a high sensitivity to construction disturbance. The alluvial sand is composed of an upper and lower deposit which are separated by the silt stratum. The thin glacial till layer overlies the rock which is at a depth of approximately 100 feet (ft) from the ground surface.

The Gateway sites are underlain by a continuous alluvial sand deposit which extends to the glacial till or rock surface. The sand contains occasional thin silt lenses located near the groundwater level. The depth to rock varies from 50 to 80 ft below the ground surface.

FOUNDATION DESIGN AND PERFORMANCE

Details concerning foundation design, construction, and performance of specific case histories will be discussed.

Westchester Financial Center

50 Main Office Tower/1-11 Martine Office Tower

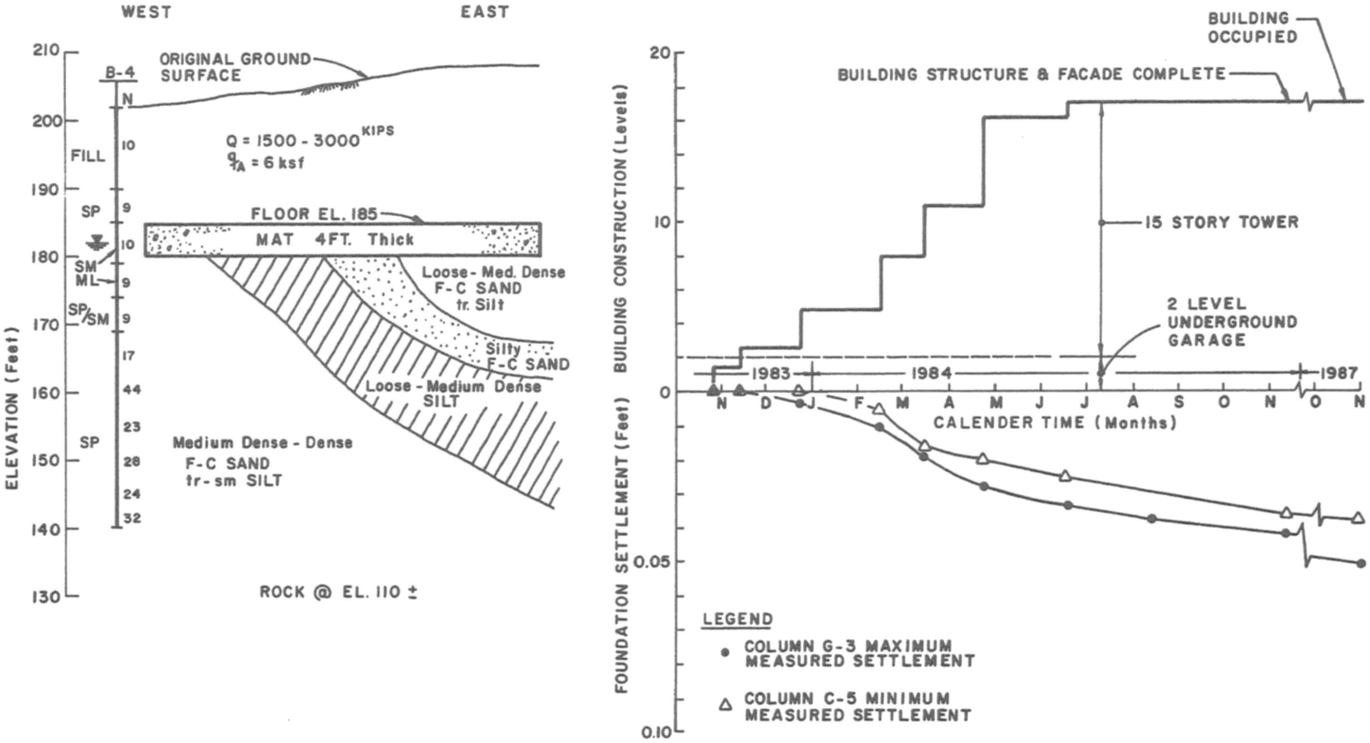
These office towers are both 15 story cast in-place concrete structures with post tensioned concrete floors and architectural facades composed of stone and glass panels. A 2 level underground garage structure is common to both buildings. The footprint area of the towers are 28,000 square feet (sf) for 50 Main and 20,000 sf for 1-11 Martine. The lowest garage floor is located at elevation (el) 185 ft and the foundation subgrade is located at el 180, approximately 20 ft below street grade. The design loads vary from 1500 to 4000 kips per column.

Subsurface Conditions

The subsurface conditions beneath the building areas are similar. The dilative silt or silty fine sand deposit was encountered at foundation subgrade and the groundwater level was at 1 to 3 ft above the bottom of the foundations. The non-plastic silt is varved with fine sand seams and was in a loose to medium dense condition. The water contents range from 22 to 40 percent, the liquid limit and plastic index are approximately 29 and 6, and the virgin compression ratio is approximately 0.06. The thickness of the silt varies from 0 to 25 ft. The lower sand deposit underlying the silt layer is in a medium dense to dense condition. The design and subsurface conditions are shown on Figures 3a and 4a.

Foundation Construction

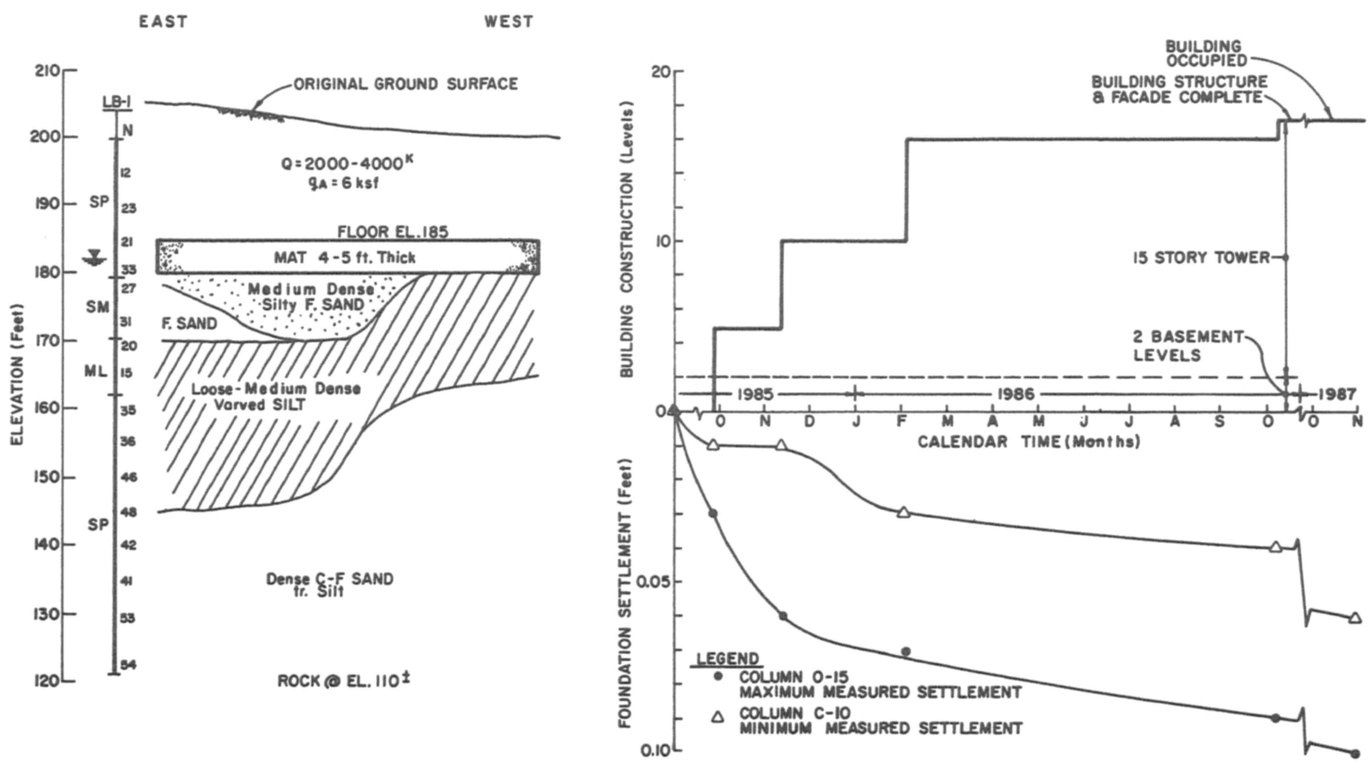
Soil improvement procedures in conjunction with 4 to 5 ft thick reinforced concrete mat foundations were used to transfer the heavy column loads to the subsoils. A majority of the foundation subgrade consisted of the saturated silt or silty fine sand soils which varied in thickness and density. The denser lower sand deposit also formed a portion of the subgrade. A mat foundation was used to span the variable subgrade and to limit differential settlement that would have occurred for a conventional spread footing system.



a. Design Conditions

b. Mat Foundation Settlement Versus Construction

FIG. 3 - 50 MAIN OFFICE TOWER



a. Design Conditions

b. Mat Foundation Settlement Versus Construction

FIG. 4 - 1-11 MARTINE OFFICE TOWER

Prior to the construction of the mat foundations, the following soil improvement procedures were accomplished to control groundwater seepage and to stabilize and confine the silt subgrade soils.

1. Overexcavation of the silt to a depth of 2 ft below the foundations.

2. Placement of a geotextile on top of the silt subgrade.

3. Placement of compacted 3/4 inch stone backfill to foundation subgrade.

A mold blade backhoe bucket was used to excavate the silt soil below foundations to minimize the disturbance of this sensitive soil. Groundwater seepage from the silt was controlled using the stone backfill and conventional pumps. Following the placement of the stone backfill, a 2 inch thick concrete "mud mat" was poured to provide a working surface for construction of the mat foundations.

Foundation Design

The foundations were designed as flexible mat foundations using the Portland Cement Association MATS computer analysis. An allowable soil bearing pressure of 6 ksf and a modulus of subgrade reaction (K) of 100 kips per cubic foot (kcf) were selected for the design of the 50 Main mat. Since the 50 Main structure was completed prior to the design of the 1-11 Martine building, the performance results from the completed building were used to refine the analysis for the design of the later structure.

Predictions

The two methods of analyses selected to estimate the settlement of the mat foundations were the D'Appolonia (1968) and the Schmertmann (1970) analyses. Both methods are applicable for layered granular soils. Since the silt exhibited non-plastic behavior it was analyzed as a cohesionless soil. The D'Appolonia approach was used with a weighted average elastic modulus for the layered soil profile. An estimation of elastic moduli of the soil layers was based on a correlation with Standard Penetration Test (SPT) N values. The Schmertmann approach uses a layered soil profile, cone penetrometer resistance, and a graphical plot of strain influence values as a function of depth to footing width. The cone penetrometer resistance was estimated using a correlation with SPT N values as a function of grain size. The predicted settlements for the 50 Main and 1-11 Martine mat foundations are presented on Table 1.

TABLE 1: Predicted Settlements - 50 Main and 1-11 Martine Mat Foundations

	D'Appolonia (1968)	Schmertmann (1970)
Total Settlement (ft) 50 Main Mat	0.20	0.09
Total Settlement (ft) 1-11 Martine Mat	0.23	0.11

Performance

Following the construction of the second level basement floor, settlement monitoring points were established on the columns. Settlement monitoring was accomplished with a high precision survey level and readings were recorded to the nearest 0.005 ft. Monitoring was accomplished through November 1987. The buildings were occupied prior to the completion of the monitoring program. Therefore, the dead and live loads were transmitted to the mat foundations. The measured foundation settlements versus building construction are presented on Figures 3b and 4b.

The measured settlement for the 50 Main foundation was 0.035 ft for exterior columns to 0.05 ft for interior columns. The ratio of average predicted total settlement to the maximum measured settlement is 3.0. The measured settlement range for the 1-11 Martine foundation was 0.06 ft for exterior columns to 0.10 ft for interior columns. The ratio of average predicted total settlement to the maximum measured settlement is 1.7. The measured results indicated that the flexible mat foundations limited the amount of differential settlement to approximately 40 percent (%) of the total measured settlement.

The average subgrade modulus computed from the measured settlements was 140 kcf for 50 Main and 75 kcf for 1-11 Martine. The selected design value was 100 kcf.

Gateway Project

Gateway I Office Tower

This office structure is an 18-story cast in-place concrete building with post tensioned floors and a glass panel facade. A one level deep basement for mechanical equipment is located below the office tower. The building has a footprint area of approximately 15,000 sf and its basement floor is at el 192. The foundation subgrade is located at el 186, approximately 20 ft below street grade. The design loads range from 1200 to 2500 kips per column.

Subsurface Conditions

The basement level is underlain by a sand deposit with occasional silt seams. The medium dense to dense sand deposit consists of fine to coarse sand with trace silt. The silt seams are approximately 3 to 12 inches thick and interspersed with fine sand lenses. Groundwater was encountered approximately 4 ft below foundations at el 182. The design and subsurface conditions are presented in Figure 5a.

Foundation Design and Construction

Soil improvement densification procedures and shallow spread foundations were used to support the office tower structure. The footings were designed for an allowable contact pressure of 6 ksf. The sand footing subgrade was densified using a 5 ton static drum weight vibratory roller.

Predictions

The Schmertmann method and a layered solution by DeBeer and Martens (1957, 1965) modified by Meyerhof (1965) were used to estimate the settlement of the spread foundations. The Meyerhof method also uses cone penetrometer resistance to estimate elastic moduli for the soil layers. The predicted settlement is presented in Table 3.

TABLE 3: Predicted Settlements - Gateway I Spread Foundations

	Meyerhof (1965)	Schmertmann (1970)
Total Settlement (ft)	0.11	0.13
Differential Settlement (ft)	0.03	0.01

Performance

Settlement points were established on the 1st floor columns and monitoring was accomplished through March 1985. Monitoring was terminated following the completion of the architectural facade at which time approximately 90 percent of the total load was transferred to the building foundations. The measured foundation settlement versus building construction is presented on Figure 5b.

The measured settlement for the office tower footings ranged from 0.055 ft for exterior columns to 0.075 ft for interior columns. The ratio of predicted total settlement to the maximum measured settlement is 1.6. The measured differential settlement was approximately equal to the predicted values.

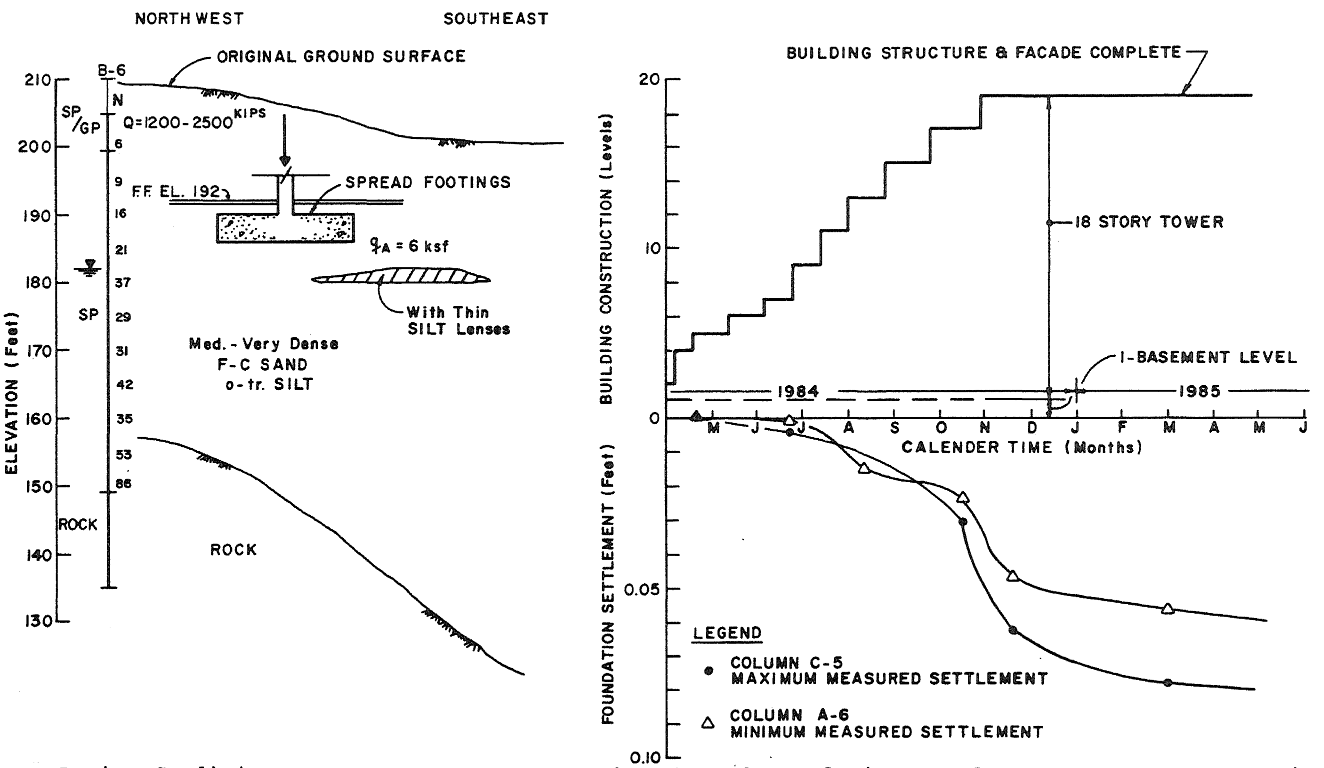
Discussion

A summary of the predicted and measured foundation settlements for the previously discussed case histories and for other building sites in the downtown area are presented in Table 4.

The maximum measured settlement occurred at the 1-11 Martine mat foundation where 0.10 ft of settlement was recorded.

The differential settlement between adjacent columns for this mat and the 50 Main mat was less than 0.04 ft for 28 ft column spacing. This amount of differential movement is considered acceptable for concrete structures. The total settlement for the remaining structures supported on shallow foundations did not exceed 0.08 ft and the differential movement between adjacent columns was equal to approximately 0.02 ft.
The predicted settlement values, based on the methods discussed in the case histories, exceeded the measured settlements by 50 to 200%. The use of SPT N values to estimate cone resistance may have led to the high predicted settlements.

The Schmertmann method appeared to provide the best estimate for the settlement of the mat foundations with a predicted to measured ratio of 1.1 to 1.8. The Meyerhof approach provided the closest approximation for estimating the settlement for the buildings supported on spread foundations with a predicted to measured ratio of 1.4 to 1.6.



a. Design Conditions
b. Spread Foundation Settlement Versus Construction
FIG. 5 - GATEWAY I OFFICE TOWER

TABLE 4: Summary of Foundation Settlement Results

Project	Building[1] Height (levels)	Foundation Type	Contact Pressure (ksf)	Predicted[2] Settlement (ft)	Measured[3] Settlement (ft)	Predicted / Measured
50 Main	17	Mat	6	0.15	0.05	3.0
1-11 Martine	17	Mat	6	0.17	0.10	1.7
Gateway I	19	Spread	6	0.12	0.08	1.5
25 Martine	14	Spread	6	0.10	0.06	1.7

1 Includes below grade levels
2 Average predicted settlement using Schmertmann (1970), D'Appolonia (1968) or Meyerhof (1965) methods of analysis
3 Maximum measured settlement

EXCAVATION DESIGN AND PERFORMANCE

Details for the design, construction and performance of temporary excavation support systems will be discussed.

Westchester Financial Center

50 Main Excavation

An excavation depth of approximately 20 ft below street grade was required to construct foundations for the 2 level underground parking structure which is common to the Westchester Center site. The soil supporting the adjacent streets and utility services needed to be retained throughout the period for construction of foundations and the underground structure. A temporary flexible retaining structure was constructed in conjunction with open cut excavation slopes to achieve the foundation subgrade. Cantilevered soldier pile and timber lagging walls were designed and constructed for exposed heights up to 13 ft. The design and subsurface conditions for the excavation adjacent to Bank Street are presented in Figure 6a.

Design and Construction

The soil parameters used for the design of the cantilevered structure are shown on the figure. A conventional earth pressure analysis (U.S. Steel, 1984) was used to determine the soldier pile size and depth of embedment. A factor of safety of 2 was used for the passive soil resistance at the toe of the soldier pile wall.

The HP 14 X 73 soldier piles were driven to the depths shown on the figure with a Vulcan 010 air hammer. The piles were spaced at 6 ft on center. As the excavation proceeded in stages, 3 inch thick by 10 inch wide timber lagging was installed behind the front face of the pile flanges to retain the soil. In areas where running sand was encountered, backpacking behind the lagging was accomplished with sand and straw hay, and the depth of unsupported excavation was reduced to one board height.

Predictions

An elastic approach assuming the soldier pile wall acts as a fixed cantilevered beam was used to estimate the maximum lateral deflection at the top of the retaining wall. The active earth pressure loading was applied in a triangular distribution assuming that the computed resultant load would be applied to a beam length equal to the exposed height of the excavation plus one half the embedment depth of the pile (U.S. Steel, 1984). The predicted elastic lateral movement at the top of the wall was 0.12 ft.

Performance

Following the installation of the soldier piles, monitoring points were established at the top of selected piles. Lateral movements were monitored throughout the excavation to foundation subgrade with optical survey equipment. Movements were recorded to the nearest 0.01 ft. The measured lateral movements versus excavation elevation are presented on Figure 6b. The measured lateral movement ranged from 0.04 to 0.17 ft. The ratio of the predicted elastic movement to the maximum measured movement is 0.71.

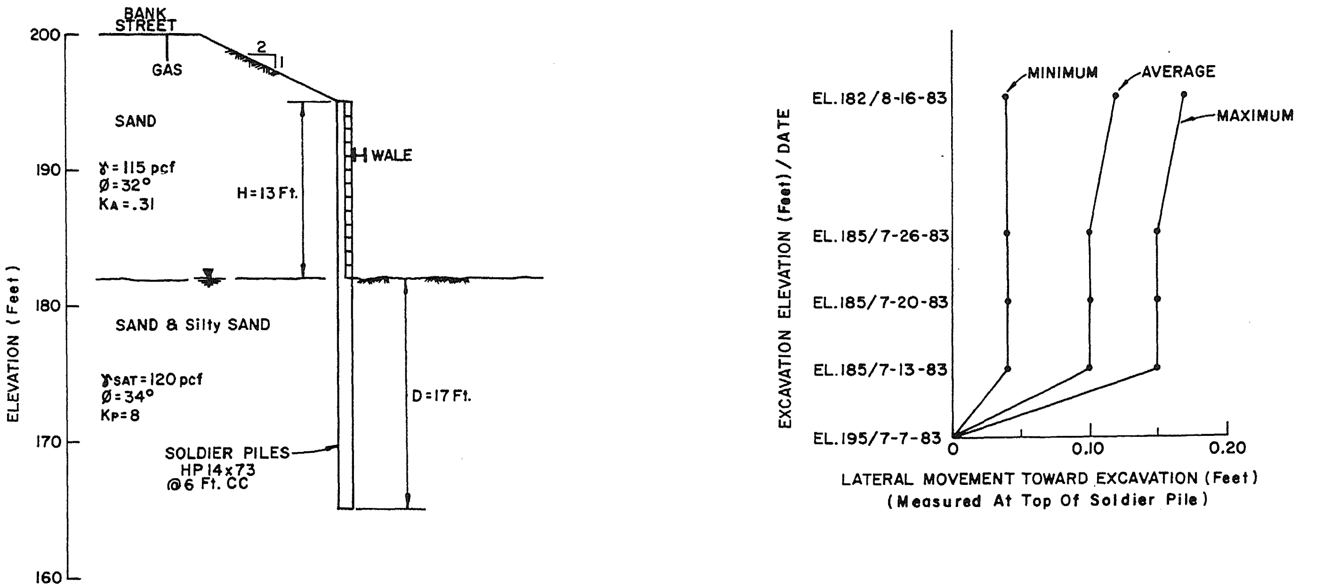
Gateway Project

Gateway I Excavation

An excavation depth of 20 ft below Hamilton Avenue was required to construct foundations for the deep basement beneath the Gateway I office tower. The contractor designed and constructed a temporary cantilevered soldier pile wall to retain the sand soil supporting the adjacent utilities and street. The exposed height of the wall was 18 ft. The design and subsurface conditions are presented in Figure 7a.

Construction

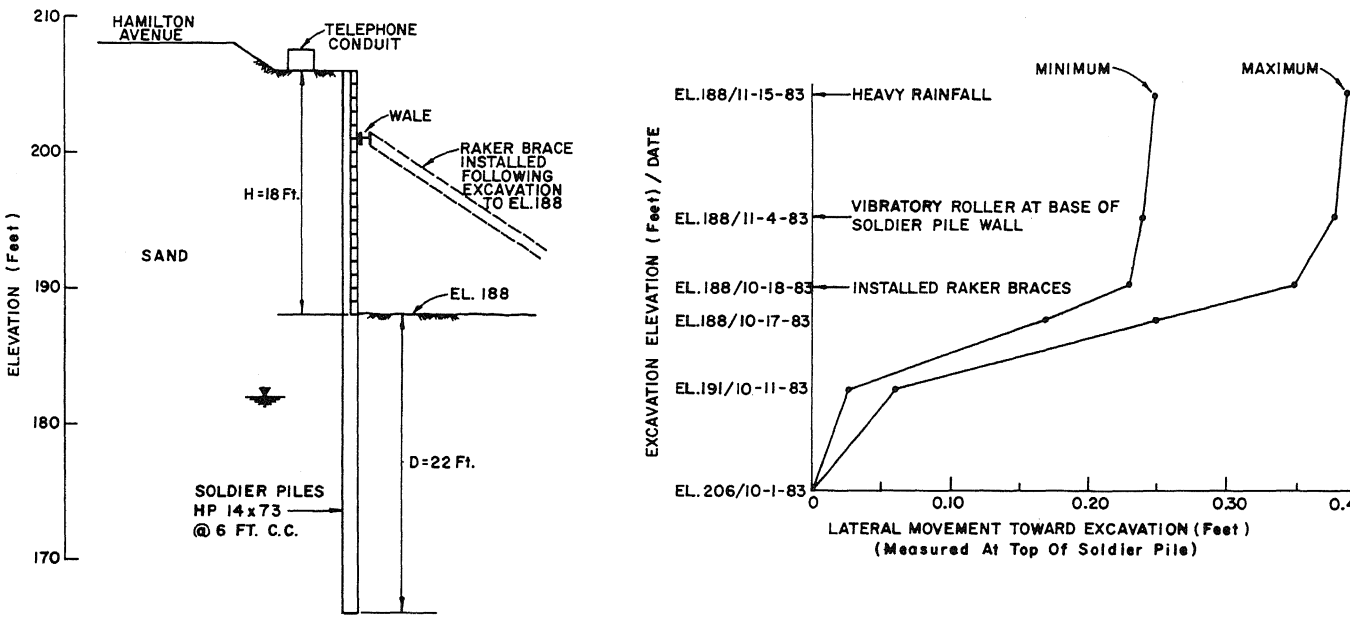
The HP 14 X 73 soldier piles were spaced at 6 ft centers and driven with a Vulcan 010 air hammer. Timber lagging was placed between the soldier piles. The excavation proceeded in stages from the top of the piles at el 206 to foundation subgrade at el 188. After the final excavation had been achieved, the cantilevered wall began to move toward the excavation at an accelerated rate. Therefore, the contractor decided to install raker braces at 12 ft centers to control the lateral movement.



a. Design Conditions

b. Lateral Movement Versus Excavation Depth

FIG. 6 - CANTILEVERED SOLDIER PILE WALL - 50 MAIN EXCAVATION



a. Design Conditions

b. Lateral Movement Versus Excavation Depth

FIG. 7 - CANTILEVERED/BRACED SOLDIER PILE WALL - GATEWAY I EXCAVATION

Predictions

The fixed elastic beam approach (as previously discussed) was used to estimate the maximum lateral movement at the top of the pile wall. A predicted elastic lateral deflection of 0.43 ft was calculated for the 18 ft high cantilevered soldier pile wall.

Performance

The contractor established monitoring points on top of the piles and recorded lateral movements as the excavation proceeded. In addition, monitoring points were established at the curb line to measure the lateral movement of the cracks in the street pavement that occurred during the excavation. The pavement cracks were located parallel to and approximately 12 ft away from the soldier pile wall. The measured lateral movements versus excavation elevation are presented in Figure 7b.

The total measured lateral movement varied from 0.25 to 0.39 ft. The ratio of predicted elastic movement for the cantilevered wall to the measured movement was 1.10. However, the installation of the raker braces limited the total lateral movement of the temporary cantilevered wall.

Gateway Project

Gateway Underground Garage Excavation

This 3 level below ground cast in-place concrete structure is located below New Street in the north area of the Gateway Project. An excavation depth of approximately 30 ft below Ferris Avenue was required to construct the garage foundations at el 165. A temporary earth retention system was required to retain the soil supporting the sidewalk, street, and utilities. The deep excavation was supported using a soil anchored soldier pile wall. The design and subsurface conditions are presented in Figure 8a.

Design and Construction

The 25 ft high soil anchored soldier pile wall was designed for a two stage construction excavation. During the first stage of excavation to the level of the wale and soil anchor, the wall was analyzed for conventional active earth pressure loading. For the second stage excavation, following the installation of the soil anchors, the wall was analyzed for approximately 2/3 of the apparent earth pressure loading. The soil anchors were designed for the full apparent earth pressure.

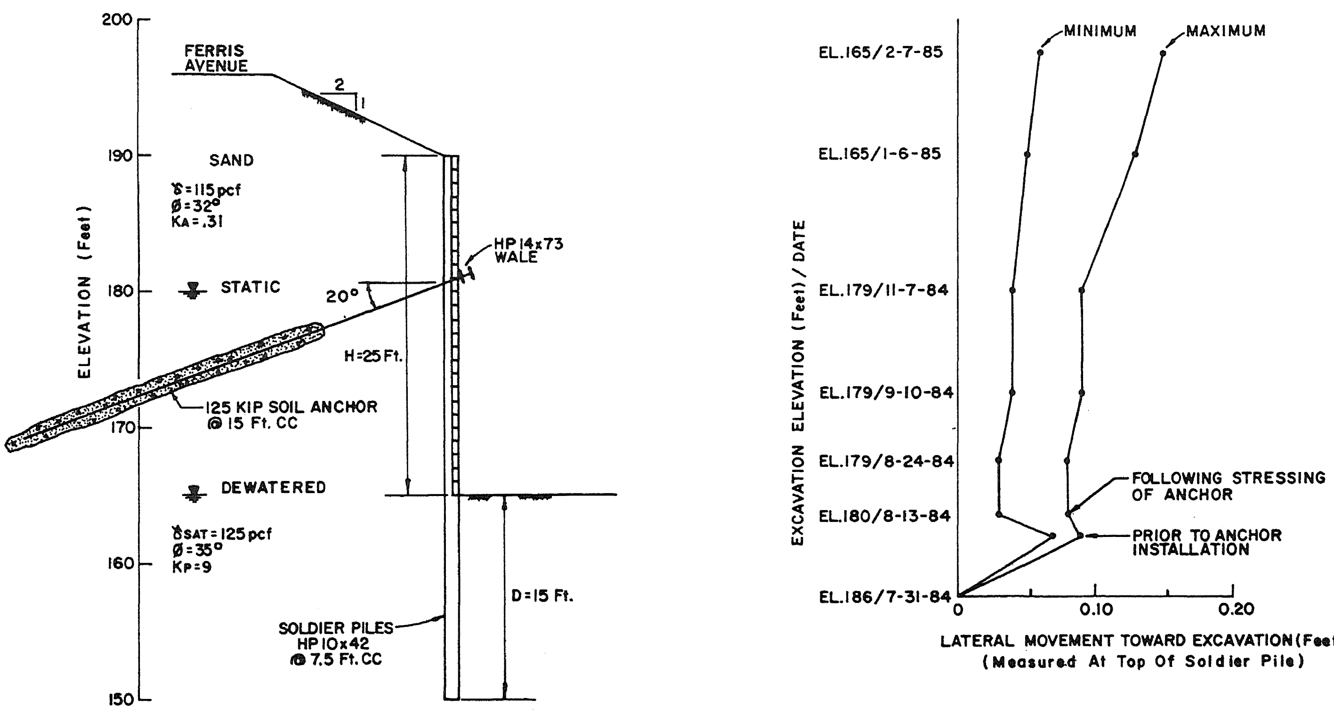
The HP 10X42 soldier piles were spaced at 7.5 ft centers and driven with an ICE vibratory hammer. Conventional timber lagging was placed behind the pile flanges, and straw hay was placed between and behind the lagging boards. The soil anchors were installed at 15 ft centers using pressure injected techniques. A 4 inch hole was drilled, cased, and washed using rotary equipment. The anchor reinforcement (four 270 ksi steel strands) was grouted in the hole using low pressure primary and high pressure secondary grout applications. A regrout tube was installed with the anchor reinforcement. The 10 ft stressing length of the anchor reinforcement was sheathed with plastic and the bond length of the anchor was approximately 25 ft. All of the anchors were prooftested to 125% of their design load and locked-off at 75% of the load.

Predictions

Since the soldier pile wall was subjected to both active soil pressure loading and concentrated point loads associated with the soil anchors, elastic superposition methods were used to estimate the lateral deflection of the wall. The predicted maximum lateral deflection at the top of the wall was 0.15 ft.

Performance

Following the installation of the piles, monitoring points were established at the top of selected piles. Monitoring was accomplished through the staged excavation sequence. Lateral movements versus excavation elevation for the Ferris Avenue wall are



a. Design Conditions

b. Lateral Movement Versus Excavation Depth

FIG. 8 - SOIL ANCHORED SOLDIER PILE WALL - GATEWAY UNDERGROUND GARAGE EXCAVATION

TABLE 5: Summary of Excavation Support Movements

Project	Retaining Structure[1]	Height[2] (ft)	Predicted Movement (ft)	Measured[3] Movement (ft)	Measured/Predicted
50 Main	Cantilevered SP	15	0.12	0.17	1.4
50 Main	Cantilevered SP	14	0.08	0.12	1.5
25 Martine	Cantilevered SP	14	0.09	0.23	2.6
Gateway I	Cantilevered/Braced SP	20	0.43	0.39	0.9
Gateway Underground	SP w/1 level of anchors	21	--	0.11	--
Gateway Underground	SP w/1 level of anchors	28	0.15	0.15	1.0
Gateway Underground	SP w/2 level of anchors	30	--	0.08	--

1 SP = Soldier pile and timber lagged wall.
2 Height = Equivalent height wall with a level ground surface at the top and bottom of the retaining wall. This equivalent height accounts for backslopes and toe berms.
3 Maximum lateral movement measured at top of retaining structure.

presented in Figure 8b. The measured lateral movement ranged from 0.06 to 0.15 ft. The ratio of predicted lateral movement to the maximum measured movement is 1.0.

Performance tests were accomplished on two soil anchors to determine the residual or permanent movement of the grouted anchor. An incremental series of load and unload cycles were performed up to 150% of the anchor design load for an 86 kip three strand anchor with a bond length of 20 ft and a 125 kip four strand anchor with a bond length of 28 ft. At 100% of their design load, the permanent (non-elastic) anchor movement was measured to be 0.026 ft for the 86 kip anchor and 0.032 ft for the 125 kip anchor. The permanent anchor movement at 100% of the design load was equal to 0.12% of the bond length of the anchor.

Discussion

A summary of the predicted and measured lateral movements for the previously discussed retaining structures and for other excavation retention systems in the downtown vicinity are presented in Table 5.

The maximum measured movement occurred at the Gateway I excavation, where 0.39 ft of lateral deflection was recorded for the cantilevered/braced 20 ft equivalent height wall. Additional lateral movement may have occurred at this site if the originally constructed cantilevered wall had not been internally braced. The 14 to 15 ft equivalent height cantilevered walls experienced movements up to 0.23 ft and the 21 to 30 ft equivalent height soil anchored walls moved up to 0.15 ft toward the excavation.

The measurements from these case histories indicate that for conventional HP soldier pile sections, the maximum equivalent cantilevered wall height is approximately 15 ft. Beyond this height, lateral movements can become excessive.

The predicted elastic movements for the cantilevered soldier pile walls were less than the maximum measured lateral movement by 40 to 160%. Construction methods and surrounding ambient conditions have led to lateral movements in excess of the estimated elastic deflection. The presence of running cohesionless sand during the lagging installation may have left voids behind the soldier pile wall. These voids sometimes extend behind the back flange of the soldier piles, thereby, significantly reducing the arching or self supporting effect of the soil between the piles. Backpacking and attempting to backfill from the top of the soldier pile wall does not usually succeed in reestablishing the natural arching capacity of the soil. In time, vibrations caused by heavy street traffic and intense rainfalls caused the voids behind the lagging to become filled with loose soil. The loose soil does not have the arching capacity of the natural dense soil. Therefore, additional soil pressures are transmitted to the soldier piles and greater than predicted lateral movements occur.

Limiting lateral movements for lagged soldier pile walls in running sand can be accomplished by the use of contact lagging attached to the front face of the soldier piles. This procedure limits the disturbance of the natural arching of the in-situ sand between the piles.

CONCLUSIONS

Through the use of field measurements, it was possible to analyze and evaluate the performance of completed building foundations and temporary earth retaining structures. Original design assumptions and methods of predicting their performance could be checked and evaluated to assist in the design and analysis of future structures.

As indicated in the discussions:

- The maximum measured total and differential settlement, 0.10 ft and 0.04 ft, respectively, was recorded at the 1-11 Martine mat foundation. This magnitude of settlement is considered to be acceptable for the concrete structures discussed.
- Measured differential settlements were observed to be less than 40% of the total measured settlement.
- The predicted settlements exceeded the maximum measured settlement by 50 to 200%.

- The Schmertmann analysis provided the best estimate for the settlement of the mat foundations and the Meyerhof method yielded the closest approximation for the settlement of spread foundations.

- The predicted elastic movements for the cantilevered soldier pile walls were less than the maximum measured lateral movements by 40 to 160%.

- Construction difficulties during the lagging installation caused by running sand conditions may have led to the increased lateral movements.

- The use of contact lagging installed on the front face of the pile flange could limit disturbance of the arching effect of the in-situ sand, thus, decreasing the potential for lateral movement of the soldier pile walls.

- For conventional HP soldier pile sections the maximum cantilevered equivalent wall height is approximately 15 ft. Beyond this height, lateral movements can become excessive.

APPENDIX I - REFERENCES

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APPENDIX II - NOTATION

The following symbols are used in this report:

D	= Depth of embedment
H	= Exposed height of wall
K	= Modulus of subgrade reaction
K_a	= Coefficient of active earth pressure
K_p	= Coefficient of passive earth pressure
N	= Standard penetration test N-value in blows per foot
Q	= Column load
q_a	= Allowable bearing pressure
ϕ	= Angle of internal friction
γ	= Total unit weight
γ_{sat}	= Saturated unit weight

APPENDIX III - CONVERSION OF UNITS

The following english units can be converted to the International System (SI) units:

1 foot (ft) = 0.3048 meters (m)
1 inch (in) = 25.4 millimeters (mm)
1 kilopound = 1000 lbf = 0.50 tons
1 kilopound (kip) = 4.448 kilonewtons
1 kilopound per square feet (ksf) = 47.88 kilo pascal (kPa)
1 pound per cubic foot (pcf) = 16.02 kilograms per cubic meter (kg/m^3)