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SYNOPSIS

This paper describes the construction of the foundation system for a 52-story apartment building between Duane Street and Thomas Street (the Tribeca Tower) in the City of New York and the design and implementation of a protection program for a 130-year old designated landmark building founded on a stone "rubble" wall foundation adjacent to the Tribeca Tower.

SITE CONDITIONS

A portion of the southern end of the Borough of Manhattan (Manhattan Island) in the City of New York is underlain by strata of saturated loose to medium dense sands. This area has a long history of building settlement resulting from the installation of displacement piles to support new structures using impact or vibratory hammers (Lacy and Gould 1985). When displacement piles were driven more than 30 years ago to support the Javits Federal Building in this area, compaction of the sand strata, produced by the vibrations from impact hammers and then from vibratory hammers mistakenly used to reduce particle velocities, caused sufficient settlement of adjacent buildings that they were purchased by the federal government and demolished. Similar, but less catastrophic settlements of existing buildings have occurred in recent years when new buildings supported on driven piles were constructed in the area.

A plan view of the site of the Tribeca Tower and the adjacent landmark building is presented in Figure 1. The site is underlain by about 20 ft of fill and 60 ft of loose to medium dense sand, beneath which is 30 ft to 40 ft of dense sand and gravel containing cobbles (glacial till) that overlies bedrock. Bedrock, which is mica schist, is encountered about 120 ft below the surface; the groundwater table is about 30 ft below the surface. Generalized subsurface conditions and the typical lengths of the piles supporting Tribeca Tower are shown in Figure 2.

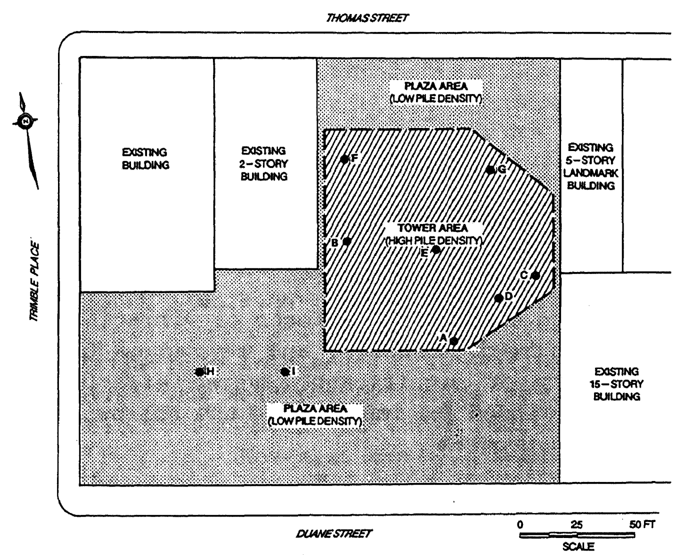

Figure 1 Site Location Plan

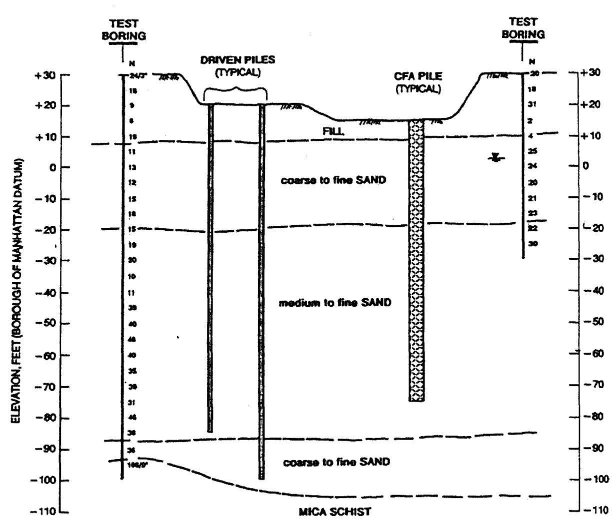

Figure 2 Generalized Subsurface Conditions

PROTECTION PLAN FOR LANDMARK BUILDING

Woodward-Clyde Consultants, Inc. (WCCI) was initially contacted by the owner/developer of the Tribeca Tower project, to develop a Protection Plan for the landmark building in response to New York City Landmarks Preservation Commission requirements. Development of the Protection Plan was expanded to include redesign of the project's foundation once it was realized by the owner/developer that the original pile foundation system was unworkable and would damage nearby structures.

The landmark building adjacent to and east of the Tribeca Tower is a five-story structure, approximately 24 ft by 79 ft in plan dimensions. The pre-construction condition of the building was documented by a detailed report complimented by photographs of the exterior and interior of the building. The landmark building contains two basements. The lowest basement level is about 8 ft below the basement level of the Tribeca Tower. The geometrical relationship between the landmark building and the proposed construction was reviewed by WCCI to evaluate the potential for damage from construction operations.

In order to observe the effects of vibrations on the landmark structure and to reduce the possibility of significant adverse effects, WCCI recommended and implemented the following monitoring program :

1. Peak particle velocities resulting from pile driving were measured continuously by seismographs in the basement of the structure. Based on our observation of the condition of the landmark building, the maximum allowable value of peak particle velocity (measured in the subbasement of the landmark building) was established at 1 inch per second (in./sec). Criteria between 1 in./sec and 2 in./sec are generally accepted as a safe for residential structures (Wiss 1981; Esrig and Ciancia 1981) although lower values are sometimes required for old buildings in poor condition. Records of vibration levels were collected and reviewed weekly when on-site activities had a low potential for causing high vibrations (i.e., excavation) and daily (or hourly) during pile installation activities.

2. Measurements of crack widths in the landmark building were made using tell-tale devices which were installed across representative existing cracks and cracks that developed on the exterior and interior walls of the landmark building. The tell-tales were monitored periodically.

3. Survey points on the outside of the walls of the landmark building and lines on the roof were established and were monitored periodically by a licensed land surveyor for horizontal and vertical movement.

4. Groundwater levels at an existing on-site observation well were monitored on a weekly basis during construction. The groundwater level during the pre-construction geotechnical investigation was measured at a depth of about 30 ft below the surface.

5. Visual inspection of the condition of the landmark building to monitor any changed condition was performed by WCCI resident engineer.

This program, known as a Protection Plan, was reviewed and approved by the New York City Landmarks Preservation Commission. A summary of the monitoring program with the frequency of measurements and allowable monitoring criteria is presented in Table 1.

During the implementation of the Protection Plan for the landmark building a total of 50 weekly or bi-weekly reports summarizing the results of the monitoring program were prepared and submitted to the Construction Manager for distribution to the New York City Landmarks Preservation Commission, the owner of the landmark building and the owner/developer of the Tribeca Tower.

ORIGINAL DESIGN FOUNDATION SYSTEM

A pile system consisting of about 1,100 pipe piles with a design capacity of 50 tons (short) was selected to support the new structure by the original geotechnical consultant to the owner/developer. 7.625 in. diameter pipe piles with a wall thickness of about 0.375 in. were to be driven open-end to depths from 100 ft to 110 ft using a Vulcan 50C impact hammer with a rated driving energy of 15,000 ft-lbs per blow. For reasons not

MONITORING PROGRAM	LOCATION	ALLOWABLE LIMIT	FREQUENCY OF MEASUREMENT During Excavation	During Pile Driving	During Completion of Construction
Vibration Measurements	Subbasement	Max peak particle velocity less than 1.0 in./sec	weekly	daily	not taken
Tell-Tale Crack Width Measurements	Exterior and interior walls	Crack width movement less than 0.01 ft (0.12 in.)	weekly	daily	monthly
Optical Survey Measurements	Exterior walls and roof	Building movement less than 0.04 ft (0.48 in.)	weekly	weekly	monthly
Groundwater Level Measurements	Observation well	Groundwater level movement less than 5 ft	weekly	weekly	not taken

Table 1 Summary of Monitoring Criteria

related to performance, the owner/developer decided to change the foundation consultants as construction was about to begin. Immediately upon reviewing the available information, WCC informed the owner/developer that significant settlement of the landmark building was likely if the open-end pipe piles were driven. After a review of alternatives, the decision was made to proceed with the foundation system previously designed and to perform a pre-construction test-driving program to investigate the likely magnitude of the potential settlement problem.

PRE-CONSTRUCTION TESTING

A 6-pile group and a 15-pile group were driven to the required final design driving resistance of 72 blows per foot [Building Code of the City of New York (Building Code) 1968] relatively far from adjacent structures to help define the settlements resulting from impact pile driving. For each test, three settlement observation points were established at distances between 4 ft and 15 ft from the center of the pile group by anchoring a 2-inch diameter pipe 4 ft into the upper natural sand and isolating the pipe from the surficial upper fill with an 8-inch diameter casing. It was assumed in analyzing the results of the tests that settlement resulting from soil compaction due to pile driving is related to the total energy (E) transmitted to the point of observation. It was further assumed that the energy arriving at any point was inversely proportional to the square of the distance from the driven pile to the observation point (R^2). Extrapolation from the results of the two tests, shown on Figure 3, suggest that the landmark building would likely settle one to two inches if all 1,100 piles were driven. Less than 0.5 inch of settlement was believed to be acceptable by the project's structural engineer. A more complete description of the pre-construction testing program and results of this program was presented by Esrig et al. (1991).

PILE LOAD TESTS FOR PIPE PILES

Two pile load tests (Pile Load Test Nos. 1 and 2) were conducted for two selected pipe piles in accordance with the requirements of the Building Code. The tested piles were designed to support a working load of 50 tons each.

During the pile load tests, the settlement of the piles was measured by gauges reading to the nearest 0.001 in. at three locations on the pile butt. Movement was also measured to

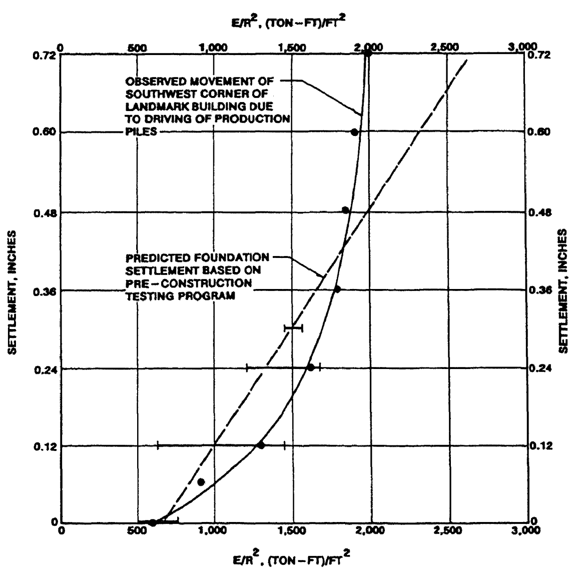

Figure 3 Settlement as a Function of Total Energy and Distance

the nearest 0.01 in. using a level. In addition to these measurements, the vertical movement of the instrumentation frame was measured to the nearest 0.01 in. using a level. Loads were applied by jacking against a dead load reaction weight for both tests. Loading increments and durations were in compliance with the Building Code.

The gross butt settlements under 100 tons of load for Pile Load Test Nos. 1 and 2 were 0.686 in. and 0.910 in., respectively. The related rebounds were calculated to be 0.672 in. and 0.771 in. Figure 4 presents a plot of the load-deflection data for Pile Load Test No. 2. Both tested piles complied with settlement criteria established by the Building Code.

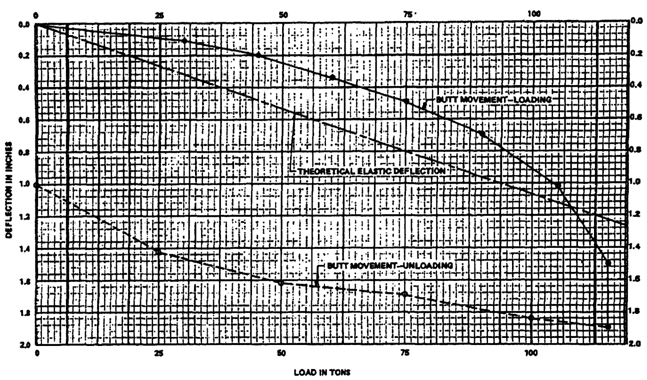

Figure 4 Pile Load Test No. 2

Based upon the satisfactory completion of the pile load tests, 50-ton capacity production piles driven open-end to a minimum final penetration resistance of 72 blows per foot were approved by the City of New York Department of Building (Building Department) for the Tribeca Tower.

PILE DRIVING

The design of the foundation system, as shown in Figure 1, required that about 800 pipe piles be driven in a relatively small central area of the site (tower area) with a high "pile density" of about one pile per 7 to 10 ft^2. The strips along Duane Street and Thomas Street (plaza areas), which represent about 70 percent of the site, has a low "pile density" of about one pile per 100 square feet. Production pile installation began in the areas of low "pile density", then moved into the center of the tower area in order that early pile driving be as far from the adjacent buildings as reasonable so that settlement problems, if any, could be identified early before becoming severe.

During the 22-week period of pile driving, when 780 open-end pipe piles were installed, the total observed vertical movement of the landmark building was between 1.0 in. and 1.2 in. The time-related portion of this movement was estimated to be between 0.2 in. and 0.3 in. The observations suggest that settlements resulting from pile driving occurred over distances in excess of 30 ft from the driven piles.

The relationship between measured settlement at the landmark building and the ratio E/R^2 resulting from the driving of about 780 pipe piles is superimposed on the pre-construction test-driving program data on Figure 3. Peak particle velocities measured inside the landmark building were less than 0.2 in./sec during production pile driving and were measured during the test-driving program at about 0.6 in./sec on the ground within 10 ft of the pile being driven and less than 0.05 in./sec at a distance of about 100 ft.

The observations of settlement during production pile driving shown on Figure 3 suggest that soil densification within the tower area, as large numbers of piles were driven, increased the energy transmission over time. This phenomenon probably occurred as soil densification reduced the local soil damping associated with soil compaction and natural soil heterogeneity, thus producing increased settlement from a given energy input at the landmark building.

During the installation of the production piles, the depth to the top of the soil plug within the open-end pile was measured in several piles. These measurements indicate that the typical height of soil column inside the open-end pipe for these piles is between 30 ft and 50 ft.

A more complete description of the pile driving program, results of the monitoring program and measured building displacements during production pile driving was presented by Esrig et al. (1991).

ENGINEERING AND SELECTION OF CFA PILES

Pre-excavation by drilling holes stabilized with drilling mud and the driving of pipe piles in those holes was considered, attempted and ultimately abandoned as too time-consuming. A review was made of available soil modification and underpinning procedures that might be appropriate if pile driving were to be completed without damaging the landmark building. These methods included compaction grouting, chemical grouting, ground freezing, underpinning by injection piles (mini piles) and underpinning by an adjustable jacking system. However, since the settlement observations clearly indicated that pile driving was causing measurable settlements due to sand compaction at distances in excess of 30 ft from the driven piles, all walls, interior columns, and the slab-on-grade would have to be supported. This was not possible for this occupied and operational building. Therefore, conversion to a low vibration, non-displacement bored pile foundation system such as continuous flight auger (CFA) piles, front-of-wall (FOW) piles or injection piles was recommended to complete the foundation. After considerable review, a CFA pile system was selected to complete foundation construction.

PILE LOAD TESTS FOR CFA PILES

A total of five 16-in. diameter, 90-ft. long, CFA piles were installed for testing; piles of this length were chosen in an effort to achieve compatibility of deformation between the pipe and CFA piles when loaded. From these five CFA piles, two were selected for pile load tests (Pile Load Test Nos. 3 and 4) in accordance with the requirements of the Building Code. Because non-standard piles were tested, both pile load tests were designed for cyclical loading/unloading and the final load increment remained in place for a minimum of 96 hours. The test piles were designed to support a working load of 75 tons (short) each. Detailed site-specific technical specifications describing the installation process and including the history, design and use of CFA piles were prepared by WCCI and were submitted to the Building Department before they would approve the use of these piles in New York City.

During the load tests, settlement of the piles was measured by gauges reading to the nearest 0.001 in. at three locations on the pile butt. Movement was also measured at three locations to the nearest 0.01 in. using an optical level. In addition to these measurements, the vertical movement of the instrumentation frame was measured at two locations to the nearest 0.01 in. using a level. Loads were applied by jacking against a dead load reaction weight for both tests. Each test consisted of two phases. During the first phase, each test pile was subjected to cyclical loading/unloading at loads ranging to 150 tons. Load increments and durations were in compliance with the applicable section of the Building Code. During the second phase of the load tests, the pile was loaded to 200 tons in six increments and then unloaded in four decrements. These quick maintained-load tests were in general compliance with quick load test methods for individual piles described by ASTM D-1143-81 (1981).

The final gross butt settlements under 150 tons for Pile Load Test Nos. 3 and 4 were 0.464 in. and 0.405 in., respectively. The related rebounds were calculated to be 0.130 in. and 0.189 in., respectively, suggesting that the net pile settlements were 0.334 in. and 0.216 in., respectively. Figure 5 presents a plot of the load-deflection data for Pile Load Test No. 4. Both tested CFA piles complied with settlement criteria established by the Building Code.

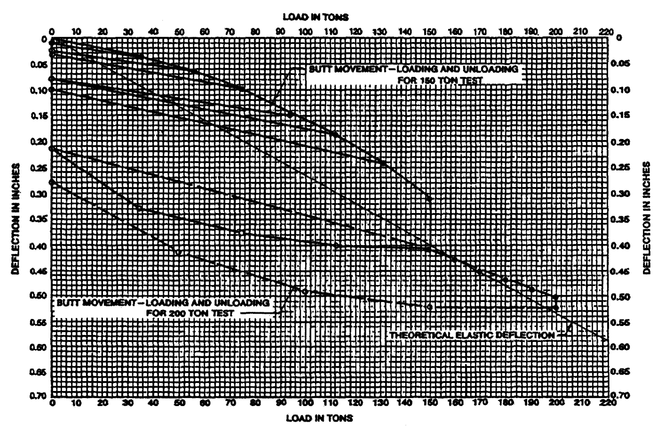

Figure 5 Pile Load Test No. 4

Based upon the satisfactory completion of the pile load tests, 75-ton capacity 16-in. diameter production CFA piles were approved for use for the Tribeca Tower. This was the first time that the Building Department approved the use of uncased CFA piles since the current Building Code was adopted in 1968.

CONTINUOUS FLIGHT AUGER (CFA) PILES

Foundation construction for the 52-story tower continued using 90 ft long, 16-in. diameter CFA piles with a design capacity of 75 tons. The extreme length of the CFA piles and relatively low capacity were required to avoid problems of differential settlement that might arise when two radically different pile systems were used to support one structure.

It was believed that the low level of vibrations associated with the installation of CFA piles would reduce future settlement of the landmark building provided that loss of ground during installation could be controlled. This was particularly important because about 230 CFA piles were to be installed within 5 ft to 30 ft of adjacent buildings. However, equipment failure during installation of several of the first CFA piles caused settlement of the landmark building and resulted in the development of detailed procedures designed to minimize lost ground.

After installation of 19 CFA piles at distances from the landmark building ranging from 4 ft to 15 ft, a measured settlement of the landmark building of 1.5 in. occurred. The immediate portion of the measured settlement was estimated to be 1 in.; the time-related portion was estimated to be 0.5 in. Most of this movement was caused by equipment failure that led to an excess volume of soil being removed from the borehole (loss of ground) during the augering operation. In several instances, full or partial reaugering due to technical problems (clogging of the injection system, movement of the bottom plug into the auger flight prior to grout pumping, loss of the bottom plug, premature setup of the grout or misconduct of the pump) was required; this caused a higher than expected volume of soil to be removed and resulted in building settlements. CFA pile installation operations were temporarily halted. In order to reduce the volume of soil removed from the boreholes, several revised technical procedures were formulated, field tested and, subsequently, adopted. A more complete description of the revised technical procedures and related inspection program was presented by Leznicki et al. (1992).

The adoption of the revised technical procedures enabled the contractor to successfully install about 210 CFA piles and to complete the foundation system.

During the production period, when the revised technical procedures and inspection program were adopted, settlements of the landmark building of up to 0.5 in. were measured with the immediate and time-related portions of the movement being equal. These movements are believed to be primarily caused by the reduction in soil strength and density (Massarsch et al. 1988; Neely 1991). In comparison, during the installation of the first 19 CFA piles, a vertical movement of the landmark building of 1.5 in. was observed. A significant portion of this movement was believed to be caused by ground loss during augering.

Our observations suggest that for CFA piles installed using the revised technical procedures, minor settlement should be expected within a radius of about 6 pile diameters or about 10 ft from the CFA pile. If loss of ground due to pile installation is occurring, the influence area may be greater than 15 pile diameters.

MOVEMENT OF LANDMARK BUILDING

During the 70-week period of construction of the foundation system for the Tribeca Tower and afterward, optical survey measurements were made of the horizontal and vertical movement of the landmark building. Three different construction activities (impact pile driving, CFA pile installation, and several periods of general site excavation) influenced the magnitude of the movement of the landmark building. For each of these operations, immediate and time-related (secondary compression) movements can be assessed.

The southwest corner of the landmark building had the largest total measured downward movement of 0.32 ft of which the time-related movement was estimated to be 0.10 ft. The highest value of the measured horizontal movement was about 0.29 ft west of the western wall of the landmark building at the roof level. Figure 6 presents the differential

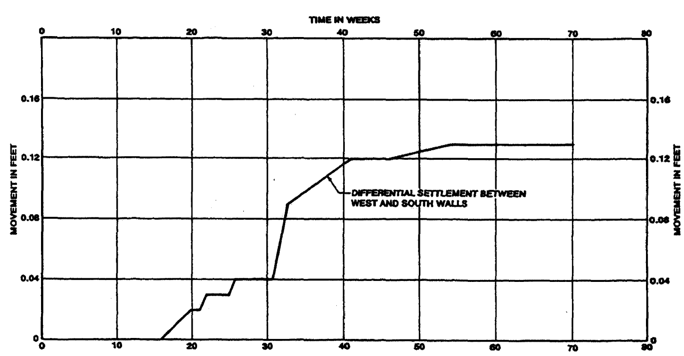

Figure 6 Differential Settlement of the Landmark Building

settlement between the west wall and the south wall of the landmark building. We were pleased to notice that despite all the imposed stresses and displacements, little or no damage was done to the adjacent structures and the landmark building preserved its structural and functional integrity.

PERFORMANCE OF TRIBECA TOWER'S FOUNDATION SYSTEM

To evaluate the long-term performance of the Tribeca Tower foundation system, an 80-week program to monitor vertical (downward) movement of selected elements of the foundation system was established. Seven optical survey points (Points A to G) were established at the basement level on selected columns supporting the 52-story tower. In addition, two optical survey points (Points H and I) were established at the basement level on two of the columns supporting the plaza. The approximate locations of these optical survey points are presented in Figure 1.

Optical survey readings were collected bi-weekly or monthly during 50 weeks of construction of the superstructure and quarterly thereafter by a licensed land surveyor. A summary of the results of the optical survey performed during the 80-week monitoring period, including the main construction milestones, is presented in Table 2.

During the monitoring period, the downward movement of the 52-story tower progressed relatively uniformly. The final measured downward movement, after an 80-week monitoring period, was between 0.60 in. and 0.84 in., with an average of 0.76 in., suggesting that the net settlement of the Tribeca Tower was between 0.4 in. and 0.6 in.
The average downward movement and differential settlement observed at the 52-story tower are presented in Figure 7. The differential settlement for the foundation system consisting of the two radically different pile systems (open-end pipe piles and CFA piles)

Weeks after Beginning of Superstructure	Selected Columns at 52-Floor Tower							Selected Columns at Plaza Area		Comments
	A	B	C	D	E	F	G	H	I	
2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2nd floor was constructed
15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
17	0.00	- 0.01	- 0.02	- 0.01	- 0.02	- 0.02	- 0.01	0.00	0.00	20th floor was constructed
19	0.00	- 0.01	- 0.02	- 0.02	- 0.02	- 0.02	- 0.01	0.00	0.00	
20	- 0.01	- 0.01	- 0.02	- 0.02	- 0.02	- 0.02	- 0.01	0.00	0.00	
21	- 0.01	- 0.01	- 0.03	- 0.02	- 0.02	- 0.02	- 0.01	0.00	0.00	
23	- 0.01	- 0.02	- 0.03	NA	- 0.02	- 0.02	- 0.02	0.00	0.00	
25	- 0.02	- 0.03	- 0.03	- 0.03	- 0.02	- 0.02	- 0.02	0.00	0.00	
32	- 0.03	- 0.05	NA	NA	- 0.05	- 0.06	- 0.03	0.00	0.00	
35	- 0.03	- 0.05	- 0.07	- 0.07	- 0.06	- 0.06	- 0.04	0.00	- 0.01	52nd floor (top floor) was constructed
42	- 0.04	- 0.07	- 0.07	- 0.07	- 0.06	- 0.06	- 0.04	- 0.01	- 0.02	
50	- 0.05	- 0.07	- 0.07	- 0.07	- 0.06	- 0.06	- 0.05	- 0.02	- 0.02	Tribeca Tower was completed
65	- 0.05	- 0.07	- 0.07	- 0.07	- 0.06	- 0.07	- 0.05	- 0.02	- 0.02	
80	- 0.05	- 0.07	- 0.07	- 0.07	- 0.06	- 0.07	- 0.05	- 0.02	- 0.02	

NA - data not available

Table 2 Downward Movement of Tribeca Tower was of considerable concern. During the 80-week monitoring period, differential settlement as much as 0.36 in. was observed. After the tower was completed, differential settlement stabilized at 0.24 in. These displacement data are believed to be very satisfactory.

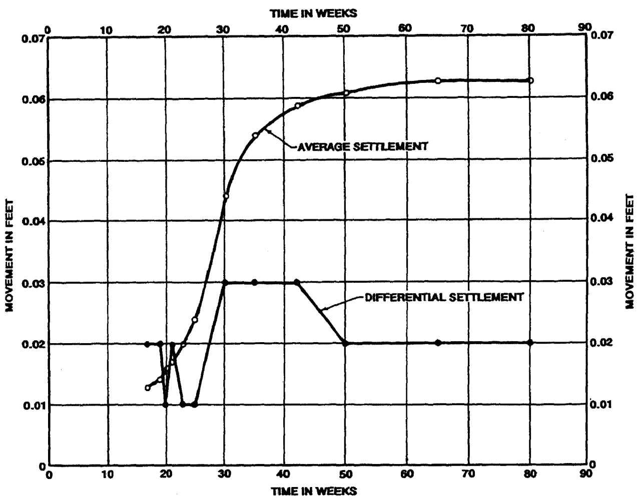

Figure 7 Average and Differential Settlement of the 52-Story Tower

CLOSING COMMENTS

1. The potential for ground loss and subsequent movement of adjacent structures exists throughout the duration of any pile installation when working in inner urban areas. To evaluate this potential problem and to select the best pile system, a site-specific detailed investigation of these concerns should be performed.

2. Densification of loose saturated sand below the existing buildings by pile driving is frequently more damaging than structural damage due to vibrations transmitted directly to the structures.

   Research is needed to describe and predict the behavior of saturated loose to medium dense sand under dynamic loading. Theoretical relationships between energy, geotechnical soil properties and volumetric soil behavior developed for seismic analysis may be useful in this research.

3. CFA piles are an effective technology for low-vibration applications. Loss of ground appears inevitable when CFA piles are installed in granular deposits even when great care is taken to reduce soil losses.

   Pile installation by contractors experienced with CFA piles and inspection by engineers knowledgeable of the potential problems that could occur and how to avoid them is essential for successful installation of this foundation system.

4. Time-related settlement appears to occur after pile installation has been completed in granular deposits and can be a significant contributor to total settlement.

5. For the foundation consisting of more than one supporting system, the potential for excessive differential settlement should be carefully evaluated during the design and foundation selection phase.

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