# Reduced Impact on Adjacent Structures Using Augered Cast-In-Place Piles

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In many cases when deep foundations are necessary to support a new structure, driven piles or other traditional support systems are inappropriate to use because of the need to protect existing structures. Often, vibrations or vibration-induced settlement that result from pile installation are major concerns. A number of projects are reviewed in which augered cast-in-place (ACIP) piles were found to be effective alternatives to other support systems. Case histories also illustrate some of the problems that can arise when using ACIP piles. A checklist is provided to assist those who are preparing to install ACIP piles and want to avoid unnecessary problems.

Pile driving often causes vibrations with peak particle velocities that are too low to structurally damage buildings but sometimes causes densification of already medium dense sands (*1*), resulting in differential settlement and damage to adjacent structures. In many of the case histories cited herein, augered cast-in-place (ACIP) piles were used as an alternative pile type after significant settlement of adjacent structures during pile driving. For example, Table 1 illustrates projects for which ACIP piles were substituted for driven piles. Designers are increasingly specifying low vibration drilled-in piles or piers for projects adjacent to vibration-sensitive structures. ACIP piles are one of the types of piles being used to address this problem.

## INSTALLATION OF ACIP PILES

An ACIP pile is installed by drilling a hole with a continuous-flight hollow-stem auger, which is plugged at the toe, to a predetermined depth. To protect adjacent structures, it is desirable to screw the augers into the ground with the least amount of soil being raised to the ground surface on the auger blades in order to avoid significant loss of ground around the pile. Fluid grout is pumped into the auger stem under sufficient pressure to eject the plug and to start forcing grout upward in the auger flights. To prevent collapse of the hole, the auger is slowly withdrawn while grout is continuously pumped. The completed grout column forms a cast-in-place pile. A single reinforcing bar can be installed through the hollow stem in advance of grouting, or 5-m-long reinforcing cages, small H-sections, or pipes can be inserted directly into the grout column while the grout is still fluid. It is necessary to provide 50 mm or more clearance. Installing longer steel members has proven difficult.

## DEVELOPMENT OF ACIP PILES

The equipment and techniques to construct ACIP piles have evolved since the pile's inception by Intrusion-Prepakt, Inc. in the 1940s (*2*).

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In its early form, the pile was constructed by placing grout through a pipe into an open augered hole. Depth was limited to about 6 m for a 305-mm diameter pile by available equipment. One modification to the method was to place coarse aggregate in the open hole after the grout pipe was inserted and allow the grout to intrude into the aggregate. Alternatively, piles were formed by grouting through a separately drilled pipe into the bottom of the augered hole as the auger was withdrawn.

Raymond Patterson was granted a patent in 1956 for these techniques and a method for placing grout through a hollow-stem auger with a removable plug at the toe of the auger, the basic methodology still in use. Augers are commonly powered by hydraulic systems capable of installing piles that are 508 mm or more in diameter to depths of more than 38 m. Equipment varies from low-torque, high-velocity turntables to high-torque, low-velocity equipment capable of virtually screwing the auger into the ground.

The size of the hollow stem increased in the 1960s with the development of pumps that could pump grout containing coarse sand aggregate. Pumps are now available that can pump grout containing pea-gravel aggregate. Hollow stems are now of sufficient size to permit installation of a central reinforcing bar without interfering with grout placement. Other, more innovations in equipment include computerized systems that monitor grout pressure and volume, auger rotation speed, torque, and depth.

ACIP piles are used for conventional foundation support elements, both vertical and battered, as underpinning piles, and as tangent soldier piles to form a continuous earth support system in conjunction with wales and tiebacks. Use of the system demands a thorough subsurface investigation to determine depth to the bearing stratum and the materials to be penetrated. Boulders or other obstructions may cause problems. Pile load-tests, usually to three times the design load, or failure, are recommended because the piles are installed to a predetermined depth without verifying resistance to penetration, as is done with driven piles. An experienced contractor, proper equipment, quality-control measures and detailed inspection of a site are requisites to successful completion of difficult projects.

ACIP piles are used throughout the United States, especially in the Midwest. They are also common in Europe and in parts of Africa. They are often referred to by the proprietary names of individual contractors who install them. The name augered cast-in-place piles is a generic name adopted by the Deep Foundations Institute's Committee on Augered Cast-In-Place Piles (*2*).

## WHY INSPECTION AND INSTALLATION METHODS ARE IMPORTANT

Table 1 and several case histories illustrate that the ACIP pile-installation method, especially within certain types of soil or

**TABLE 1 Settlement of Structures During Pile Driving and Following Substitution of ACIP Piles**

| Project | | Soil | | Initial Pile | | | ACIP PILE | | |
|---|---|---|---|---|---|---|---|---|---|
| No. | Location | Type | Ave N | Type | Distance From Structure | Settlement of Structure | Dia. mm | Distance From Structure | Settlement of Structure |
| 1 | Manhattan | m-c sand tr.silt | 20 | TPT[2] | 3m+ | 6mm | 406 | 1m | 0 |
| 2 | Southern Brooklyn,NY | m-f sand | 23 | 273mm pipe | 8m | 70mm | 356 | 3m | 38mm |
| 3 | Manhattan | m sand tr.silt | 20 | TPT[2] | 8m | 0 | 406 | 1m | 0 |
| 4 | Southern Nassau Co. NY[1] | m-f sand | 25 | - | - | - | 508 | 1.5m | 0 |
| 5 | Brooklyn,NY | m-f sand | 20 | wood piles | 0.7m+ | 6mm | 305 | 0.5m | 0 |
| 6 | West Brooklyn | m-f sand | 25 45 | 14HP73 | 1m | 61mm | 356 | 1m | 0 |
| 7 | Tri-beca Manhattan | f-c sand tr.silt | 25 | 203mm O.E.[3] | 6m+ most 1.5m few | 70mm | 406 | 1.2m | 25mm |

Notes: (1) ACIP piles were specified due to vibration sensitive equipment in an adjacent building.
(2) TPT - Tapered Pile Tip pile used by Underpinning and Foundation Constructors, Inc., Maspeth, NY.
(3) O.E. - Open-end pipe pile

groundwater conditions, can result in the detrimental settlement of adjacent structures. Methods for reducing detrimental settlement, by minimizing the amount of soil removed during insertion of the auger, for example, are described in two of the case histories.

The continuous integrity of an ACIP pile usually is controlled by monitoring the volume of grout pumped for each 1.5 m of auger withdrawal. It is common to require 110 percent of the neat volume of the augered hole with reduced rates, but continuous pumping for the last 1.5 to 3 m of auger withdrawal below the ground surface—even though grout is usually flowing onto the ground from around the auger flights. Pile installation also appears to be more successful if the auger is withdrawn slowly and continuously instead of removed more quickly in interrupted increments. Grout pressure in the hose close to the auger is monitored to identify quickly any blockage of flow and to ensure continuity from pile to pile.

## PILE CAPACITY

A range of pile capacities has been verified through load tests. Load tests performed on ACIP piles of varying diameter (*3*) are illustrated in Figure 1. Only three of these piles were loaded to soil failure. Load tests commonly were carried to three times the planned design load. Design loads generally ranged from 222 to 534 kN (25 to 60 tons) for 305-mm to 406-mm piles. The sites shown in this figure are located in areas underlain by glacial-outwash sands of similar gradation and density, and most of the sites are located near New York City.

The 356-mm-diameter ACIP piles at the Southern Brooklyn site penetrated 15 m into the bearing stratum and were loaded to 1334 kN (150 tons) or three times the design load without failure. At a nearby site, a different contractor test-loaded two piles to failure that extended only 12 m into the bearing stratum at only 801- and 1112-kN (90- and 125-ton) loads. A 15-m pile was then load-tested at the second site to failure at 979 kN (110 tons), a lower load than was sustained at the adjacent first site. The authors concluded from a comparison of results for these two sites (Figure 1) that a contractor's technique for installing this pile type is more

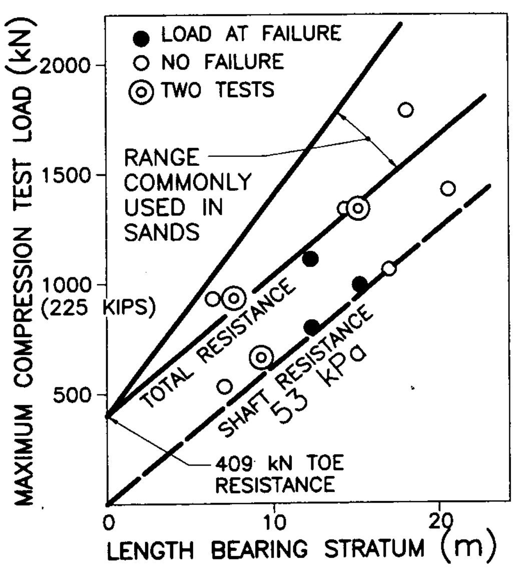


**FIGURE 1 ACIP pile load test results.**

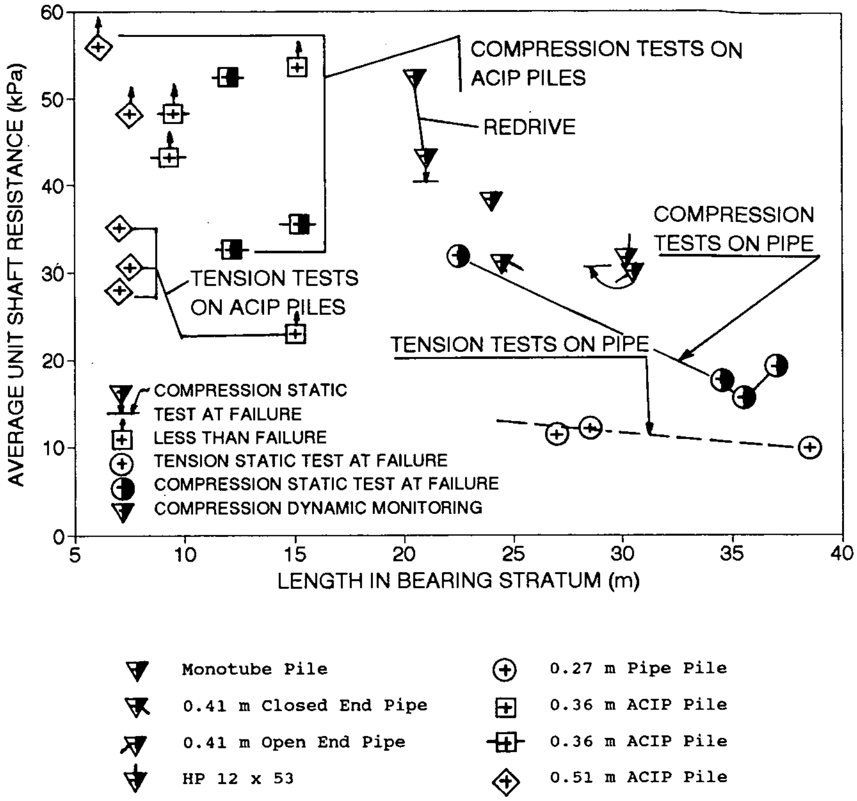


**FIGURE 2 Average unit shaft resistance.**

important than small variations in soil density and the grain size of the soil in which the pile is installed.

Unit shaft resistance for ACIP piles (*3*) is illustrated in Figure 2, which plots shaft resistance over length of pile in the bearing stratum. Results of both compression and tension tests are plotted. Unit shaft resistance in compression is estimated after subtracting toe bearing, which is determined either from telltales used during the load test or by estimating the static toe-bearing capacity based on soil parameters and the shape of the static load-test curve. Developed unit-shaft resistance varied for ACIP piles between 31 to 54 kPa (0.7 and 1.2 ksf). A number of the developed unit-shaft resistances were for piles that did not reach failure during the load test. Figure 2 also shows unit-shaft resistances for other types of prismatic piles. The pipe piles developed significantly lower unit-shaft resistances than ACIP piles, H-piles, or monotubes at failure, both in tension and compression. There is a definite trend of reduced unit-shaft friction with increasing depth.

Analysis of pile load-tests using telltales to measure the pile-toe movement indicates that nearly all of the pile capacity is in shaft resistance at service loads. As the load is increased to two and three times the design load, most of the increase is initially supported in shaft resistance. The amount of resistance provided by the toe of the pile increases with increasing load and over the duration of the load test.

## CASE HISTORIES

The following case histories are presented to illustrate both how ACIP piles can be advantageous to a project and, in some cases, present limitations and dangers with inappropriate use.

### Protecting an Old Sewer

A developer proposed to construct an 18-story office building with two basements adjacent to a street containing a 75-year-old, unreinforced concrete interceptor sewer with a 3.5-m inside diameter, as shown in Figure 3. A parking garage was planned for construction directly over the sewer. The sewer had a history of collapses in sections within a few kilometers of the site. A preconstruction walk-through inspection confirmed what our finite-element studies predicted, namely, the presence of tension cracks in the sewer crown and at invert. Piles were considered necessary to transfer building loads to a bearing stratum below the sewer, but driven piles were considered inappropriate because vibration from installation could potentially damage the sewer. Relocating or upgrading the sewer would be too expensive; since the sewer flowed full and was occasionally under surcharge all but about 4 hr per week, the sewer could not be taken out of service or damaged.

The subsurface profile shown in Figure 3 is composed of about 3 m of heterogeneous fill over a deep deposit of interlayered medium-compact to compact-fine sand, varved silt and clay. The fine sands were particularly troublesome because of their tendency to run under unbalanced hydrostatic conditions. Groundwater was about 4 ft below the level of the proposed basement slab.

An instrumentation program that included five slope-inclinometer casings, settlement points, and pore-pressure monitoring devices was instituted at the beginning of construction to provide an early-warning system that would permit time for corrective measures that could safeguard the sewer. Inclinometers were to be read and

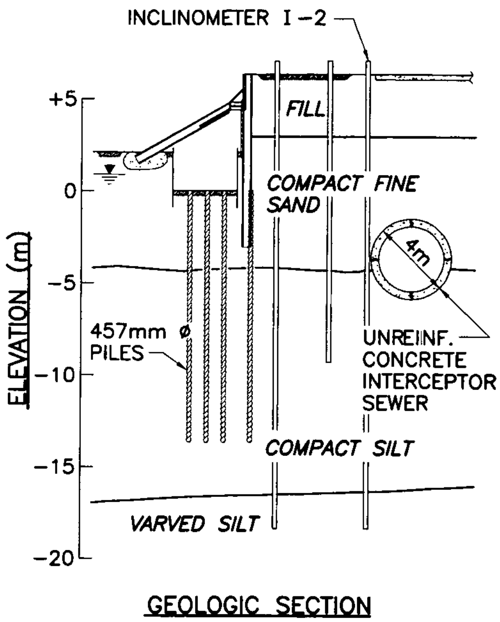


**FIGURE 3 Protecting an old sewer.**

the data reduced daily. The ACIP pile load-test program led to the selection of a 457-mm diameter, 667-kN (75-ton) design load pile installed to a depth of about 7.6 m below the sewer.

Production ACIP-pile installation began in mid-June of 1987. The contractor used a low-torque, high-rotation-rate gear box to power the augers. We observed that the augers were rotating as many as 20 times per advance of 1 auger pitch. Volumes of spoil during auger insertion were difficult to estimate because of the fluidity of the material as it came off the augers. However, monitoring of grout takes showed volumes averaging about two times the nominal volume of the augered hole, indicating significant loss of soil during auger insertion.

Figure 4 is a plot of horizontal ground movements over time for one of the initial inclinometers. The graph, annotated with construction events, shows that significant, rapid ground movements were observed shortly after production-pile installation began. The piezometers and settlement points showed no response. However, on the basis of field observations of the work and movements indicated by slope inclinometers, the contractor was ordered to stop work.

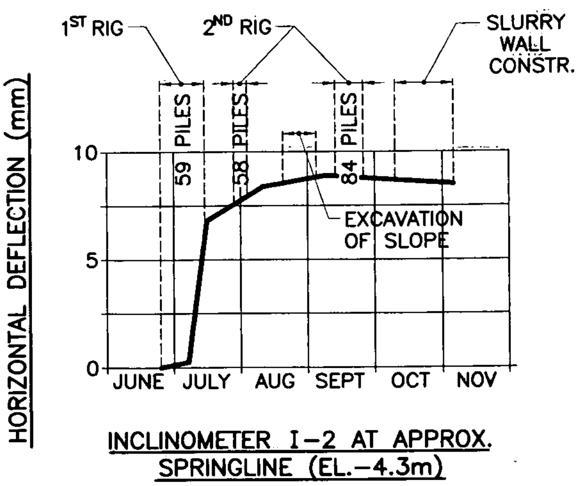


**FIGURE 4 Deflection of inclinometer at springline.**

After reviewing the work, the contractors, owner, and engineers met, and the contractor arranged to provide a turntable with a higher rated torque of 43 kN.m (32,000 foot-pounds) with the ability to throttle down the auger rotation rate. Rotation was limited to two or fewer revolutions of the auger per advance into the ground equal to the length of one pitch of the flight. We observed an immediate reduction in the volume of grout taken in each pile to about 60 percent above the nominal volume of the hole drilled by the auger. Note that these volumes included spoil grout at the surface after removal of the auger from the ground, as well as any volume that may have remained in the hoses after completion of the pile. Typical total grout take on most projects is targeted in the vicinity of 25 to 40 percent over the nominal volume.

During the first work stoppage, additional instrumentation was installed to monitor earth movements. When work resumed, a small amount of additional movement was observed. We judged that no further reduction in the ratio of auger rotation per pitch to penetration could be made and concluded that the unbalanced earth pressure resulting from the retained slope was contributing to the movements. The slope was cut back, creep ceased, and production piling was resumed. Negligible movements were observed thereafter.

Later when a second contract was let for the garage piles, the contract stipulated a minimum torque requirement for the rig, and a maximum rotation rate during advance and withdrawal of the augers. ACIP piles of similar size and capacity were installed on both sides of the sewer, and movements in small fractions of inches were observed. A postconstruction condition survey of the sewer revealed no further cracking or damage. However, in order to permit long-term monitoring of the sewer, provisions were made in the design to install permanent sewer settlement points through accessible boxes in the garage floor. Piles for both structures were designed for full loss of soil support in an influence zone adjacent to the sewer in the event of any future collapse.

### Tri-Beca, Manhattan

A 52-story residential tower planned in a congested site (shown in Figure 5) was bordered by two buildings, two and six stories in height, respectively. The site is underlain by more than 30 m of medium compact-fine to medium sand as illustrated in Figure 6. The original foundation consisted of 178-mm outer diameter, open-ended pipe piles, with a design capacity of 445 kN (50 tons), driven to depths of about 30 m. Vibration in medium-dense sand causes soil densification and settlement. A test program was carried out to observe the effects of vibrations on these two buildings. The engineers for this building estimated that the landmark six-story building might settle between 13 and 25 mm and the two-story building might settle 25 to 50 mm when production piles were driven, based on extrapolation from field measurements of ground settlement from driving the test piles. The estimate for the two-story building was higher because more piles were concentrated closer to it.

Pile driving began at the beginning of 1989. Piles farthest from the existing buildings were driven first. As soon as pile driving

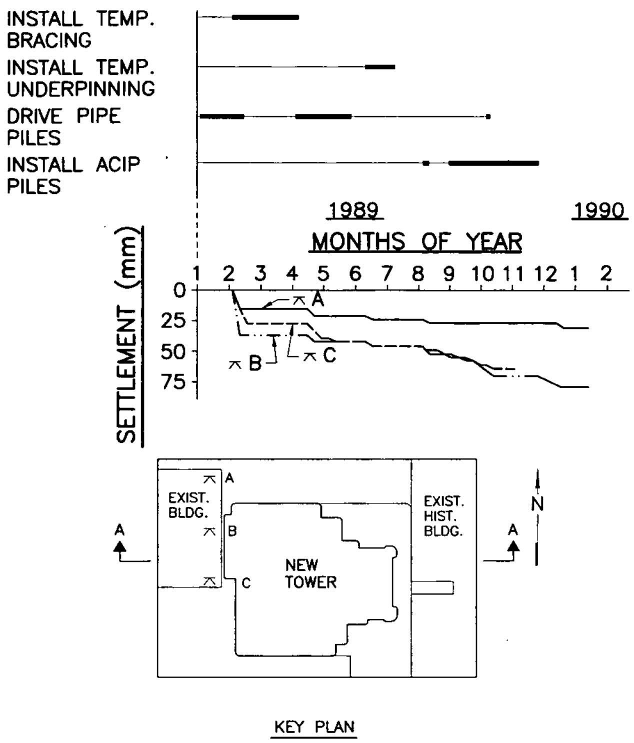


**FIGURE 5 Tri-Beca building settlement.**

moved to the center of the site between the two buildings, the two-story building started settling rapidly reaching as much as 38 mm, as shown in Figure 5. The contractor attempted to pre-auger the first 15 m and insert the pile to this depth without driving. There appeared to be no benefit to this procedure as most of the driving resistance and the vibrations generated occurred in the bottom 9 m of driving. Although both buildings experienced settlement, the discussions herein will concentrate on the two-story building, being the one the authors were primarily involved in.

All pile driving was stopped, and the contractor installed raker bracing that was not completed until April when the pile driving resumed. However, additional building settlement resulted and

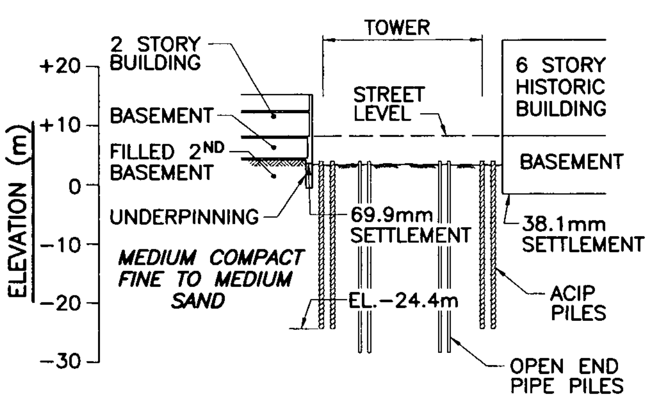


**FIGURE 6 Tri-Beca geologic section A-A.**

pile driving was stopped in June when the contractor proposed that underpinning be installed. The underpinning consisted of a series of jacks installed both above the existing footings and within the wall, using a continuously reinforced concrete beam to minimize further settlement by lifting the building as the footing settled more. All the settlements shown are net settlements for the building. Settlement of the footings was significantly more than what is shown after installation of the underpinning. It was then the developer's engineers decided to complete the foundations using ACIP piles.

A test-pile program was implemented in August, and production proceeded from September through November. Even though the underpinning jacks reduced settlement, it could be seen that the south end of the building experienced an additional 25 mm of settlement when ACIP piles were being installed. This illustrates that ACIP piles may not, at some sites, eliminate adjacent settlement due to loss of soil into the auger blades. However, we concluded that if the originally planned pipe piles instead of the ACIP piles had been driven in close proximity to this building, much more building settlement would have resulted, probably requiring demolition of the building. The building was occupied during construction, and repairs to the building were made.

## Southern Brooklyn

Foundations installed for a sludge-degritter building for a sewage treatment plant consisted of 273-mm outer diameter, closed-end pipe piles. Test piles demonstrated it was necessary to extend these piles 37 m below the excavation level to obtain an allowable capacity of 418 kN (47 tons). As the piles were being installed, monitoring indicated that the adjacent aeration tanks were settling differentially. After approximately two-thirds of the piles had been installed and settlement of the corner of the aeration tank closest to the new construction had reached about 70 mm, it was decided to stop driving the piles and complete the foundations by installing ACIP piles. The relative locations of the structures are shown in Figure 7. Figure 8 is a cross section through the structures and shows typical piles.

Most of the remaining piles to be installed were in close proximity to the aeration tanks, as it had been decided when settlement was first noted to concentrate pile driving as far away from the aeration tanks as possible, to not delay the project. Figure 7 shows the progressive settlement of three locations on adjacent structures during pile driving between March and June 1983. The aeration tanks were emptied during the early part of the pile-driving program, when settlement was first observed. Subsequently, the ACIP test-pile program was carried out. During August, the tanks were refilled, and in October production ACIP-pile installation proceeded.

An additional 38 mm of settlement occurred during the ACIP-pile installation. Figure 9 illustrates the settlement that occurred at varying distances from the expansion joint in the middle of the aeration tank, with the 37-m distance or monitoring point being at the excavation. Curves 1, 2, 3, and 4 show a progressive increase in the aeration tank's settlement during pile driving and 17 days after the piles were driven. Note that significant settlement extended more than 30 m from the edge of excavation during pile driving. Also shown in Figure 9 is additional incremental settlement that occurred during ACIP-pile installation, which at the edge of excavation equaled the maximum settlement that occurred

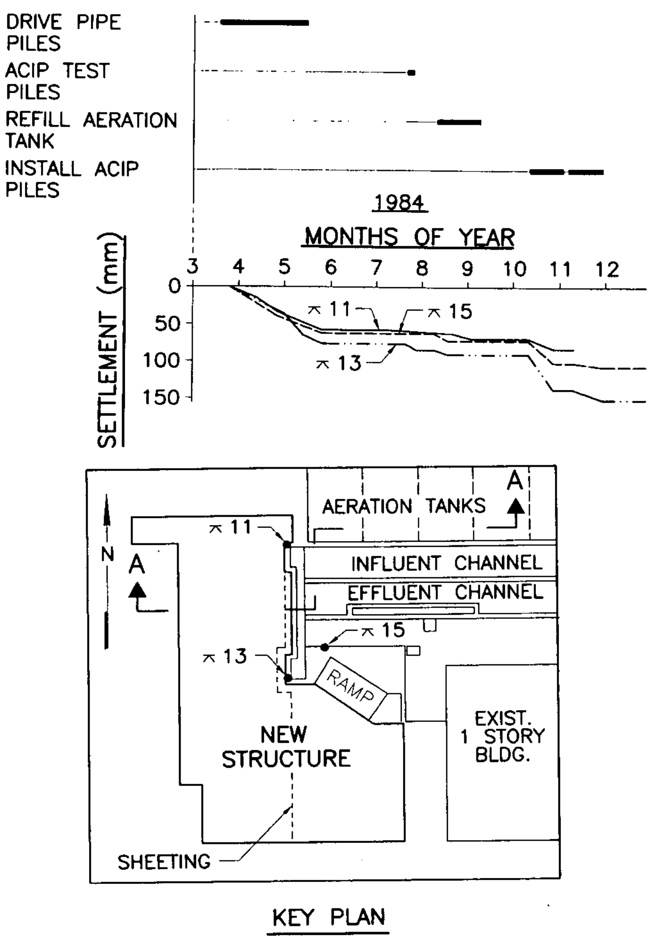


FIGURE 7 South Brooklyn tank settlement.

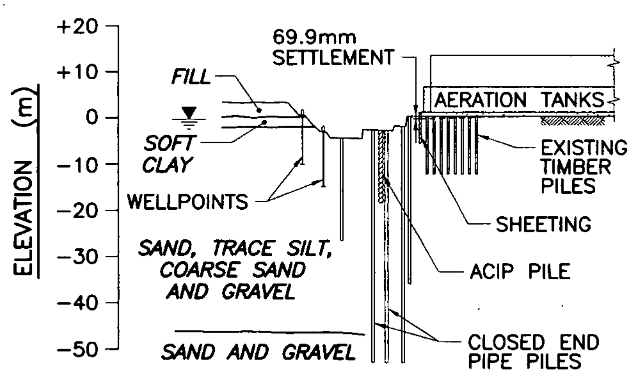


FIGURE 8 South Brooklyn geologic section A-A.

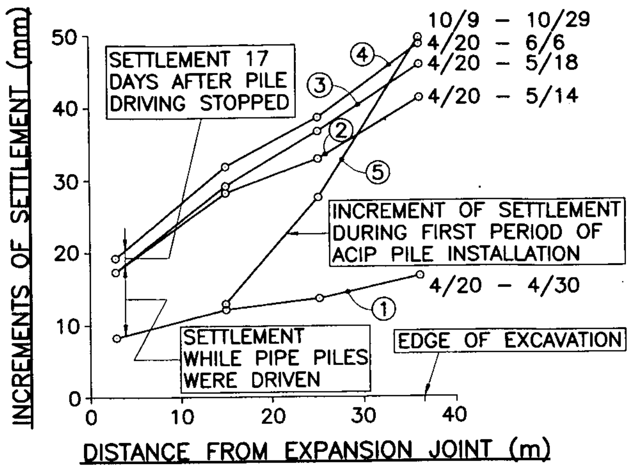


FIGURE 9 South Brooklyn building settlement versus distance.

during pile driving. The magnitude of settlement decreased much more rapidly with increasing distance from the excavation.

Horizontal movement of cantilever steel sheeting installed through and along the edge of the excavation was observed during installation of the ACIP piles. This sheeting also had moved much more during pile driving. As the sheeting had been placed very close to the permanent structure, with the intent of casting the wall against the sheeting, the sheeting's movement became a critical consideration. The top of the sheeting moved as much as 50 mm, while at the level of excavation subgrade it moved about 25 mm during ACIP-pile installation.

In the immediate vicinity of the sheeting, it was noted that the amount of sand that had accumulated around the auger blades at excavation subgrade during auger advance was substantially greater than elsewhere on site. Approximate estimates of the cone of soil indicated the volume of sand being removed from the hole was 50 to 70 percent of the neat volume of the augered hole. Grout pumped was generally in excess of 200 percent of the neat volume, including grout waste. Over the rest of the site, volumes of soil removed during auger insertion and grout pumped during auger extraction were 15 percent and 125 to 140 percent, respectively. The volumes of soil and grout resulted from the high passive-pressure stresses present below subgrade in front of the cantilever sheeting. The stresses caused greater amounts of sand to press into the loose materials riding on the auger blades as the auger penetrated to the design elevation. As a result, the removal of extra materials, as each pile was progressively installed along the wall, resulted in a release of the passive pressure and deflection of the sheeting before the passive resistance was restored. If ACIP piles are to be used next to sheeting, and deflection of the sheeting is important, they should be used cautiously.

## West Brooklyn

Figure 10 illustrates another site where H-piles were being driven to depths as much as 46 m immediately adjacent to an existing 2-story structure. The adjacent structure was supported on short timber piles that were supported by a layer of loose, fine sand, underlain by a medium-compact sand. The piles generally penetrated just a short distance into the medium-compact sand. Vibrations from pile driving densified the sand and caused one corner

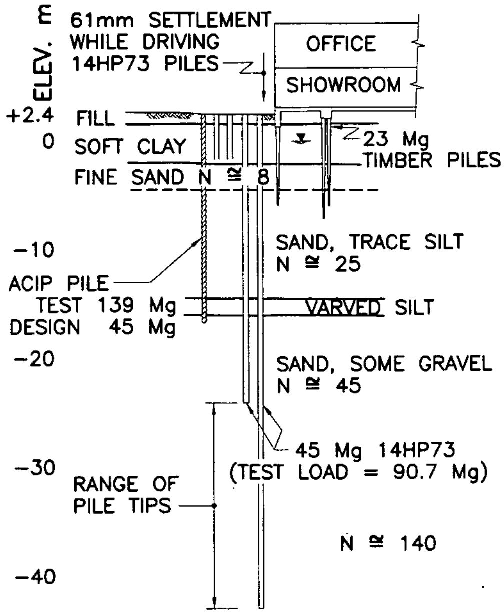


**FIGURE 10 West Brooklyn geologic section.**

of the 2-story structure to settle as much as 61 mm. Pile driving was stopped, and ACIP piles were substituted for H-piles during the remainder of the project. The ACIP piles extended into the bearing stratum for a design load of 445 kN (50 tons). They were successfully load-tested to a 1.33 MN (150-ton) capacity. On this project the contractor was paid on a unit-price basis for the H-piles. As a result, the much shorter ACIP piles resulted in a net savings to the project, even though additional pile load-testing had to be performed.

## Building Inside a Graving Dock

A new residential and recreational waterfront development was proposed on the site of a former shipyard. The centerpiece of the project included two 27-story residences to be constructed inside a graving dock originally built of timber at the turn of the century and rebuilt with concrete during the 1940s and 1950s.

Figure 11 shows a transverse section through the graving dock. In the plan, the graving dock is surrounded by land on the inboard end and along its south side. Its north side is adjacent to a pier, separated from open water by about 9 m of fill and a somewhat deteriorated steel sheetpile wall that was to be repaired. The floor of the graving dock is about 8 m below local Mean Sea Level. Below the floor of the graving dock, the subsurface profile consists of approximately 1.8 to 2.4 m of alluvium and gravel fill, underlain by interbedded, hard red clay and very compact sand of the Potomac Group (Cretaceous Age).

In some locations within the graving dock, standard sampler penetration resistances in the Cretaceous layer were in excess of 100 blows per 0.3 m, whereas at other locations penetration resistances in the upper sand were as low as 22 blows per 0.3 m. During the subsurface exploration program, fine sand layers were encountered in the Cretaceous layer, which ran up in the borehole casing despite the use of weighted drill fluid. Piezometric levels in the upper portion of the Cretaceous layer were observed to be

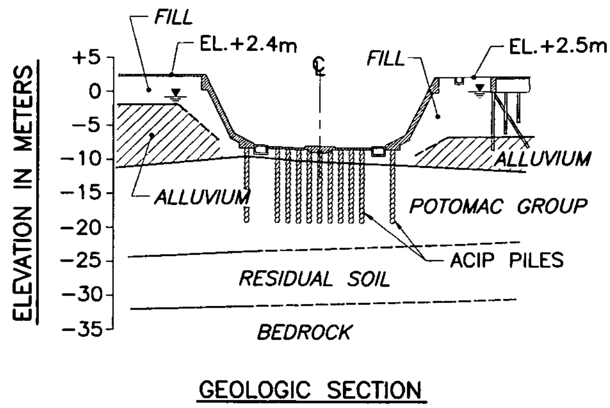


**FIGURE 11 Building in a graving dock.**

2 to 4.6 m above the graving dock floor. Water flowed out of borings made through the graving dock floor.

Consideration was given to the use of spread foundations or driven concrete piles to support the new structures. A support scheme was developed to protect the graving dock walls. Spread foundations would have required substantial dewatering effort, and the project appeared to be moving toward the use of high-capacity precast concrete piles. An experienced local contractor proposed ACIP piles as an alternative, at a substantial savings to the owner. Concerns were voiced about their use, particularly because of the presence of running sand and the effective "artesian" condition. The contractor proposed to perform load tests demonstrating the capability of ACIP piles to achieve 150-tons working capacity, at no cost to the owner if the tests failed. Demonstration tests were performed outside the graving dock, where hydrostatic groundwater conditions existed. The contractor elected to install 406-mm-diameter piles only 4.6 to 6.1 m into the Cretaceous layer; the contractor then successfully loaded the piles to twice the required capacity.

Before starting production work, a load-test program was undertaken inside the graving dock to develop installation criteria, including penetration into the bearing stratum, volume of grout, pumping pressures, and allowable capacity. Both tension and compression test piles were installed. Reinforcing typically consisted of a single #18 bar installed with centralizers. A limited number of piles received short cages for additional lateral capacity. The load tests, which included installation of telltales to the pile toe, proved valuable. Not only were the tests helpful in developing acceptability criteria, they gave some indication of problems the contractor would encounter.

The load-test program proved that compression piles with only 130-tons working load could be installed at a penetration of 11.6 m into the Cretaceous layer. Grout volume was established about 60 to 70 percent over nominal pile volume, pumped at a gage pressure of about 517 to 689 kPa (75 to 100 psi). Our analysis of the load-test data suggested that ultimate unit-shaft resistance amounted to as much as 192 kPa (4 ksf) and ultimate end bearing may have been as high as 6.7 MPa (140 ksf), probably due in part to the proximity to the residual soil layer. Tension capacity was estimated to be 356-kN (40-tons) working load.

Two significant problems besides reduction in capacity occurred, both relating to the unbalanced hydrostatic condition. After grouting of the pile was completed and the auger withdrawn, water was observed flowing upward around the perimeter of the pile, or through the pile itself around the central reinforcing bar. Often, the condition was not apparent until a couple of hours after installation. Both conditions were considered cause for rejection of the pile. Keeping the reinforcing bar 1 to 2 ft above the toe of the pile and deleting a grout additive tended to reduce the likelihood of seepage but was not always successful. The contractor had much less control in cases where water seeped upward around the pile, and a number of piles had to be redrilled, raising the cost.

The project demonstrated that ACIP piles could be installed to surprisingly high capacities for short penetrations into Cretaceous soil. However, the case history also illustrates that unbalanced groundwater can be a severe hazard. The two structures are now near completion, and the piles are performing as designed.

## CONCLUSIONS

Augered cast-in-place piles have proven successful alternatives to driven piles for a number of projects; however, ACIP piles are not without their own limitations and dangers. The case histories we discussed highlight problems that can arise when using ACIP piles, although ACIP-pile projects can be trouble free. We offer these recommendations and cautions:

1. Understand the subsurface conditions. Artesian water, boulders and other obstructions, and running ground can be especially troublesome.
2. Use an experienced contractor with experienced lead field personnel. ACIP piles require specialized equipment, techniques, and quality control measures.
3. Consider whether specifications need to be project specific. The reasons for rejecting unacceptable piles should be clear. Nondestructive integrity-test methods are available for evaluating the pile cross section, but they do not determine capacity. It is least disruptive to the project, and in the long run probably most economical, to redrill piles immediately instead of relying on remote testing or additional load testing of piles.
4. Provide full-time resident inspection of the work. The inspector's responsibility and authority must be clear. Require detailed records of grout pressure and volume of flow for each increment of auger withdrawal.
5. Monitor adjacent structures just as would be done for driven piles or excavation. Unbalanced earth pressures can be troublesome.
6. Conduct a pile load-test program in advance of production work. Production should follow the same installation procedure and use the same equipment and materials that were used for the test.

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