High Capacity Pinpiles for Structural Underpinning

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ABSTRACT

The benefits of pinpiling have been exploited in the USA during the last two decades. The demands and skills of American practice have led to developments of sufficient importance to now distinguish it sharply from aspects of the original European technology. This paper provides summary details from almost 50 projects and reviews in greater depth four major case histories. These particular projects illustrate major features about new developments in design, construction and performance in unique applications.

1. INTRODUCTION

A number of recent papers have described the state of practice in pinpile technology in the United States (Bruce, 1988; 1989; 1991; Bruce and Gemme 1992). These papers demonstrated that pinpiles - known elsewhere in the world as minipiles or micropiles, but defined generically as small diameter cast-in-place bored piles -had arrived relatively late in the United States, probably some 20 years or so after their European genesis in the early fifties. However, the growth in their use had been dramatic, as the specialist geotechnical construction industry responded to the demands of renovation, remediation and redevelopment, largely in urban and industrial locales. These reviews also detected that the pinpile market in the United States was generating a special identity for itself as a result of its ongoing pursuit of progressively higher unit capacities in a wide and complex range of subsurface conditions.

This paper presents brief reviews of four selected case histories of pinpile projects conducted by the author's company alone, in the few years since the first survey. To begin with, it should be noted (Table 2) that 20 pinpile projects can be cited in the two years from early 1989, compared to the total of 25 projects listed in the previous 11 years (Table 1). This is clear evidence of the rapidly increasing use of the technique, especially in the older cities of the East Coast (Bruce 1988a). Equally noteworthy is the routine use of individual pile working loads appreciably higher than in earlier years, or elsewhere in the world. Whereas the test loads of 200 tonnes in fine sands in Brooklyn, 340 tonnes in sand near Portland, OR, or 320 tonnes in poor rock in Pittsburgh (Table 2) are, admittedly, exceptional, normal working loads of 70-100 tonnes in soils, and higher in rock are now fairly routine.

This expansion in application, and growth in working loads, have been supported by evidence from numerous exhaustive test programs, executed in a consistently routine and scientific manner as an integral part of each commercial project. In most cases the test piles have been loaded in incre-

Location	Location/Application for foundations being underpinned	Ground conditions	Installation conditions	Load (tons) working/test	Number of production piles	Total length installed (ft)	Individual length (ft) typical/range	Nominal drilled dia. in bond zone (inches)	Construction data Reinforcement & casing	Grouting
Appolo, PA	New tank in existing wastewater treatment plant	Loose fill with concrete obstructions over clay over med. to v. dense sands with silt and gravel	Plant measured 38' × 48' in plan. Maximum headroom 18'	10/20	45	1350	30	5	#11 rebar in lower 20' + 5" casing in upper 15'	Type I w = 0.5 Maximum press 100 psi
Brookgreen Gardens, SC	Supported masts of suspended net forming 'natural' aviary in swamp, with minimal damage to environment	Loose sands & organics over medium-dense sand	Natural cypress swamp	55 generally (15 for centre pile)	25	1174	30 to 35 for verticals 55 for rakers	5	#9 rebar full length 5" casing in upper 20'	Type I w = 0.5 Maximum press. 120 psi
Nevile Island, PA	Existing dust collector structure on rapidly compacting soil	Loose fill over compact sand and gravel	10' to 16' headroom	30/60	32	928	29	5	#9 rebar in lower 16' 5" casing in upper 20'	Type I w = 0.5 Maximum press. 100 psi
Providence, R.I.	Test to assess viability of underpinning existing granite block seawall	Quay, bearing on silt, sand and fill overlying sandstone bedrock	Open air	55/110	1 (Test)	65	65	6	5" casing for 57'	Type I w = 0.45 Gravity fill
Trafford, PA	New printing press in existing building	Loose cinder fill over silty clay and weathered shale bedrock	14' headroom	10/20	20	720	36	5	5" casing full length	Type II w = 0.5 Maximum press. 100 psi
Warwick, NY	Existing gymnasium building (use of preloaded piles)	Loose sandy silt and glacial till becoming denser with depth	Minimum headroom 20'	27.5/55	62	4030	65	5	2 No 0.6" dia. strands (for preloading) 5" casing in upper 40'	Type I w = 0.45 Maximum press. 120 psi
Monessen, PA	Existing operating coke battery, emission control facility	Fill over clayey sand and gravel	19' to 25' headroom	50/100 (comp) 35 or (tension) 45	102	6330	55 and 65	5	#7 rebar full length 5" casing for all except lower 10'	Type II w = 0.45 Maximum press. 100 psi
Mobile, AL	Two existing sodium hydroxide storage tanks under which wood piles had failed	Soft organic silt and clay over dense sand with gravel	Very restricted access. 8' to 15' headroom. Caustic chemical spills	34 54	171 7	9600 400	56 Range 46 to 60	5 6⅝	5" or 6⅝" for full length except lower 8'	Type I w = 0.5 Maximum press. 80 psi
Burgettstown, PA	Existing gantry runway	Slag, silty sandy clay & shales over sandstone and limestone	Maximum headroom 24'. Soil saturated with sulphuric acid	10	20	640	32	4 (for 3' rock socket)	3½" casing full length	Type II w = 0.45 Maximum press. 40 psi
Dunbar, PA bedrock	Addition to water treatment plant	Fill over fine sand and sandstone	Open air	45	7	179	26 (Range 25 to 26)	5	#6 rebar for lower 10' 5" casing for upper 20'	Type III w = 0.45 Gravity pressure
Pittsburgh, PA	Existing structure adjacent to deep excavation	Fill and fine alluvials over dense sands & gravels with trace silt	Open air	50	21	630	30	5	5" casing for upper 20'	Type I w = 0.5 Maximum press. 60 psi
Pittsburgh, PA	Existing parking garage	Fill and alluvials over sandstone/siltstone bedrock	8' to 10' headroom	55	46	1980	43 (Range 38 to 44)	5	5" casing to rock head	Type I w = 0.45 Gravity pressure
Aliquippa, PA	New emission control building at existing coke battery	Slag fill over dense sand & gravel	25' headroom	50/100 (comp) 75/150 (tension)	31 8	2170 600	70 75	5 5	#6 rebar for lower 25' 5" casing for upper 50'	Type I w = 0.45 Maximum press. 120 psi
Jeanette, PA	New machine in existing building	Fill, silts and clay over bedrock	20' headroom	Total of 150 tons of structural weight supported	27	945	35	5½	5½" casing full depth	Type I w = 0.45 Gravity pressure
Appolo, PA	New nuclear power structure in existing building	Loose fills with clay over medium sands with gravel	20' headroom	10	24	552	23	5½	#7 rebar full depth 5½" casing for upper 18'	Type III w = 0.45 Maximum press. 150 psi
Marion, IN	Existing body stamping plant	Silty sand over rock	18' headroom	60	24	1680	70	7	7" casing for upper 50' #11 rebar for lower 25'	Type I w = 0.45 Maximum press. 50 psi
Alcoa, TN	Data not available									
Washington DC	Existing structure at Castle Building, Smithsonian Institute	Fill over dense sands with gravel	Very restrictive access and hole entry conditions	50/100	21	1580	75 (range 69 to 77)	5½	#11 rebar full depth 5½" casing between footing and bond zone	Type I w = 0.5 Maximum press. 140 psi
Pittsburgh, PA	Restoration of existing Timber Court Building	Sands and gravels, over sandstone bedrock	10' headroom	50	15	1050	70	5½	5½" casing full length	Type I w = 0.45 Gravity pressure
Warren Co., NJ	New bridge pier	Karstic limestone with voids and gouge	Open air, small area	100/224	24	1889	78 (Range 44 to 200)	8½	7" casing full length	Type III w = 0.5 Maximum press. 50 psi
Kingsport, TN	New storage tank in existing building	Silts and sands over limestone	11' headroom	40/80	115	4025	35	5½	#8 rebar in lower 15' 5½" casing to bedrock	Type I w = 0.45 Gravity pressure
Boylston St. Boston, MA	Existing building being redeveloped	Soft fills and organics over medium dense sand	Minimum headroom 8' in very restrictive basement conditions	40/92 (comp) 12/27 (tension)	252	7070	27	5½	#8 rebar full length 5½" casing in upper 19'	Type II w = 0.5 Maximum press. 60 psi
Ann Street Pittsburgh, PA	To support new soldier beams for new retaining wall	Weathered shale & sandstone over competent sandstone	Open air	45/68 (comp) 8/12 (lateral)	86	1000	11.5	6	#11 high strength rebar full length	Type I w = 0.45 Gravity pressure
Coney Island, NY	Rehabilitation of existing repair shop	Fill and organic silt over dense sands	Minimum headroom 8'. Very difficult access in fully operational facility	15/30 and 30/60	2300 1900	80 500 85 500	35 45	6⅝ 7⅝	#6 rebar full length #9 rebar full length	Type I w = 0.45 Maximum press. 60 psi
Cleveland, OH	New addition to existing control building	Slag fill and soft silty clay over shale bedrock	Open air but difficult access due to ongoing steel plant operations	60	45	6390	142	6½ (for 5' rock socket)	7" casing to rock head #8 rebar for 5" rock socket & 10' into casing.	Type I w = 0.45 Gravity pressure

TABLE 1 -- Some pinpile projects executed in the U.S.A. by Nicholson, 1979-88 (Bruce, 1988) (Imperial Units)

APPLICATION	GROUND CONDITIONS	INSTALLATION CONDITIONS	LOAD (TONS) WORKING/TEST	NUMBER OF PRODUCTION PILES	TOTAL LENGTH INSTALLED	TYPICAL PILE LENGTH	NOMINAL DRILL DIA OF BOND ZONE	INTERIOR PILE COMPOSITION	NOTES
Foundations for new electric furnace in existing building	Slag fill, soft silty clay over shale bedrock	Low headroom	35/-	12	1500'	125'	5-1/2"	5-1/2" casing plus 1-1-3/8" rebar in bond zone	———
Underpinning of existing building being redeveloped	Fill and soft clay over bouldery till	Restricted access, with 8' minimum headroom	60/120	97	4850'	50'	7"	2" high yield rebar	Two tests to 120 t: total def. = 0.222" permanent def. = 0.050
Support to spread footings of existing pipe bridge, already settled 18"	Stiff clay	Very difficult access to and under bridge	12-1/2 / 25	4	280'	70'	5-1/2"	5-1/2" casing	———
Foundations for new bridge abutment	Silty soil over Karstic limestone	Overhead power lines	77/235	48	1026'	23-80'	8-1/2"	9-5/8" casing to rock, 7" casing full length	At test load: total def. = 0.250" permanent def. = 0.032
Support for foundations in operational paper mill	Fills over shales with quartzitic seams	Access through doorways; minimum 12' headroom	55/130	33	1320'	40'	5-1/2"	5-1/2" casing	———
Support for column foundations to permit excavation of hazardous waste	Low level radioactive fill and silty clay with rock fragments over siltstone and shale	Interior of operating steel mill, minimum 10' headroom	60/-	20	800'	40'	5-1/2"	5-1/2" casing	———
Foundations for exterior stairway for existing psychiatric center	loose fill overlying very compact glacial till	14x14' access to interior courtyard	5-38/ 20-75	103	2760'	25-32'	8"	1/2"-1" rebar	At test load: total def. = 0.110" permanent def. = 0.030
Foundations for new river bridge	15' of alluvials and weathered rock over granite/gneiss	Good access, unlimited headroom	70/140	72	1901'	25'	7"	7" casing plus 1-3/8" high yield rebar in bond zone	———
Replacement foundations for 60-year-old delicate bascule bridge	River bed silts and clays over 30' dense fine-medium sands	Most from bridge deck, 4 from very limited access/headroom	50/100	52	5200'	100'	7"	7" casing plus 1-3/8" high yield rebar in bond zone	See text.
Underpinning of footings subjected to additional loads in operational detergent factory	23' clay over various medium-fine sands with interbedded clays	Very restricted access, minimum 8-10' headroom	50/100	143	5291'	37'	7-5/8"	1-3/8" dia high yield rebar, plus 7" casing in upper 10' for lateral resistance	Routine use of postgrouting to enhance soil/grout bond.
Intensive underpinning of historic 5 story building threatened by deterioration of original wood piles	Peats and clayey silt over silty fine sands	Very restricted access, 8-10' headroom	70/250	121	4500'	35-40'	7"	15-20' of upper 7" casing with 20-25' of 1-3/8" rebar in bond zone	Described in 'Civil Engineering' in Dec. 1990.
Test program for underpinning of historic building	Sands and silts over fine and silty dense sands	Through concrete footings in old structure, headroom as low as 8'	70/135-150	4 (test)	140	30-40	5-1/2"	10-30' casing, bond lengths with full length 1-1/4" rebar	Excellent test data including use of post grouting.
Supporting existing columns of operating hospital to permit adjacent and ulterior excavation	Siltstone, shale, claystone	Interior of very sensitive building with 10' headroom	125/325	18	775'	43'	7"	7" casing	See text.
Temporary and permanent piles to support overhead roadway	Fine-medium glacial sands with silts and clays	Reasonable access, 16'+ headroom	60-100/ 120-250	77	4250'	50-60'	7"	7" casing	Excellent vertical lateral testing, with postgrouting.
Foundations for pipe bridge foundations for mill expansion	20' soils over 15' shale and limestone	Through and around existing foundations	100/-	172	6020'	35'	6"	7" casing to rock, 3 ea 1-3/4" rebar in bond zone	———
New column foundations for fire damaged church	Clay over karstic limestone	Difficult access, low headroom	20-35/-	50	1750'	35'	5-1/2"	5-1/2" casing to rock, 1" rebar in bond zone	———
Test pile for underpinning of major transport facility	Clayey fill over sanitary landfill over loose sand and stiff clay	Unrestricted	-/80	1	130'	130'	5"	5" casing to top of bond zone, 3 ea 5/8" dia rebars below	At failure load: total def. = 1.060" permanent def. = 0.09
Underpinning for new and existing foundations for historic, massive building being refurbished	Fill over various alluvial fine-medium sands with cobbly/clayey horizons	Existing basement with 8-17' headroom in 3 areas	75/150	609	37500'	51-58'	7"	25-30' casing plus 25' of 1-3/8" rebar in bond zone	See text.
Foundation for pedestrian bridge	Backfill over claystone	20' headroom within 18' of existing structure	75/-	12	540'	45'	6-1/2"	5-1/2" casing	———
Foundation for temporary highway bridge	25' of alluvials and weakened material over schist	Unrestricted	75/-	4	100'	25'	7"	7" casing	———

TABLE 2 -- Some pinpile projects executed in the U.S.A. by Nicholson, 1988-90 (Bruce, 1992) (Imperial Units)

mental, cyclic fashion, so permitting movements to be partitioned into elastic and permanent components. Analysis of the former provides indications of load transfer mechanisms within the pile, whereas examination of the latter essentially permits the behavior of the founding medium to be investigated. This approach has significantly aided the understanding of how the ancillary ground treatment technique of post grouting works to enhance pile performance (Bruce, 1989a).

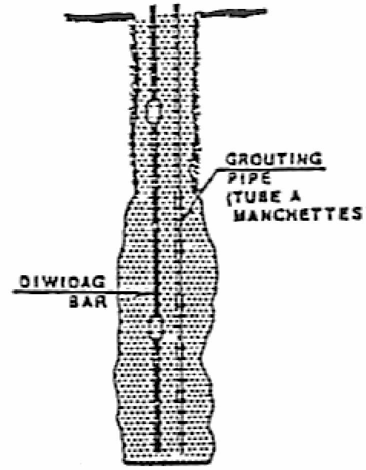

FIGURE 1. GEWI pinpile (After Mascardi, 1982)

The principle of enhancing grout-soil bond by postgrouting has long been expounded (Jones and Turner, 1980; Herbst, 1982; Mascardi, 1982; Bruce, 1991). Grouts are injected through a separate grouting tube (Figure 1) or through the steel core pipe itself (Figure 2) at regular intervals by a double packer. The grouting improves soil-grout bond (Figure 3), and may increase the nominal pile cross section, especially in weaker soil layers. PTI (1986) indicates for ground anchors an enhancement potential of 20-50% in both cohesive and cohesionless soils. Postgrouting can

- allow higher loads to be sustained for similar pile dimensions;
- allow equal loads to be sustained for reduced pile dimensions;
- allow "failed" piles to be "repaired" to safely reach target working loads.

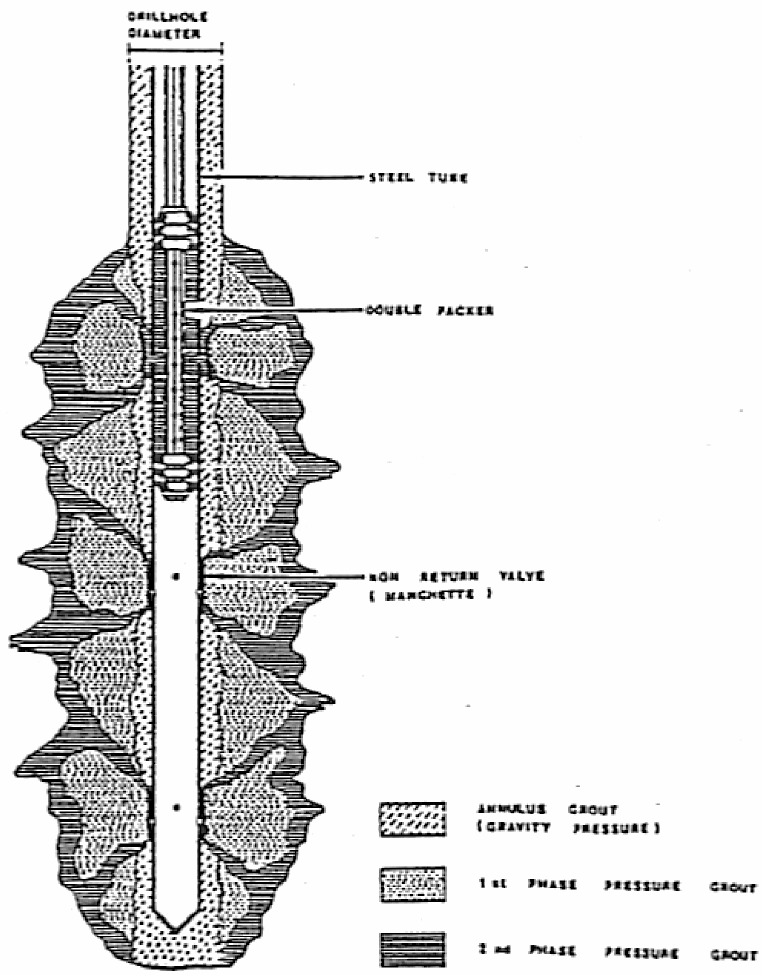

FIGURE 2. Concept of repeated postgrouting through the steel core to enlarge effective grouted diameter (Mascardi, 1982).

More experience is also being obtained with the concept of preloaded piles. The aim is to cause compression of the pile (elastic and permanent) prior to connecting it to the structure to be underpinned. In this way, further settlement of the structure is not required to "activate"

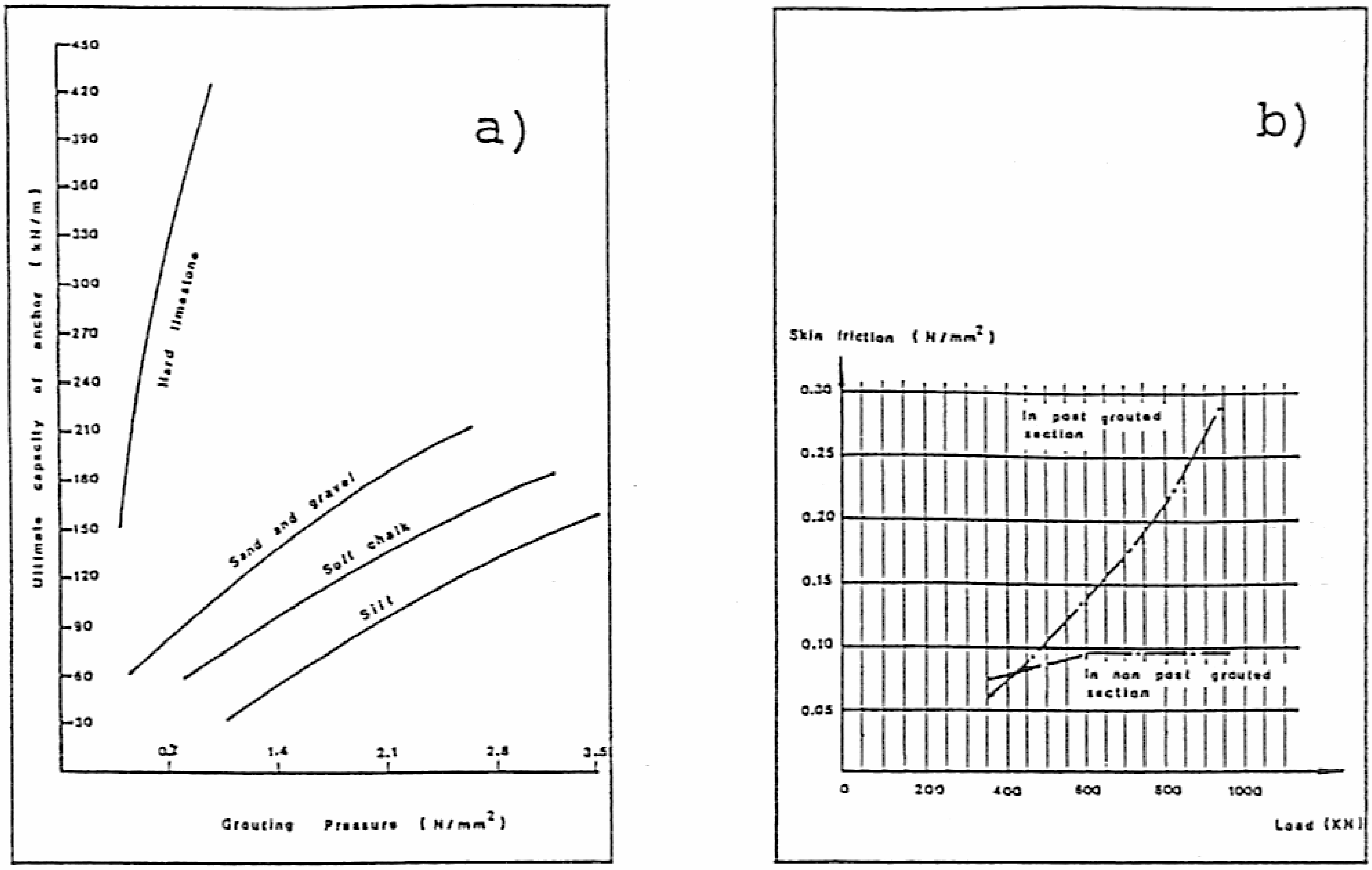

FIGURE 3. a) Influence of grouting pressure on ultimate load holding capacity (Littlejohn and Bruce, 1977). b) Effect of postgrouting on skin friction (Herbst, 1982).

the pile, and so this technique is well suited to the problems of supporting extremely delicate structures. This was used recently in the underpinning of the Pocomoke River Bridge, MD, an old and very sensitive structure (Bruce, et al., 1990). Incidentally, this principle is also of extreme potential in the seismic retrofit of bridge structures. Pier footings are often supported on soft soils by piles. Applying extra vertical load through conventional prestressed rock anchors bearing on the footing may therefore cause overloading of these pile foundations. On the other hand, prestressing has the benefit of reducing the amplitude of any "rocking" motions which may develop during a seismic event. The recently developed NCA-Pile^{SM} solves both problems by providing a very stiff prestressed pile/anchor sytstem which will act equally efficiently in tension and compression, without putting any extra vertical load on the existing foundation system.

In addition, it should be recorded that much recent attention has been focused on the performance of pinpiles when subjected to lateral loading. The impressive results from laboratory testing (Brown, et al., 1988) have been supported by recent field test programs (Table 2) at Brooklyn and Augusta: provided casing is left in place through the upper, typically "soft" horizons, pinpile performance is excellent.

2. REVIEW OF ILLUSTRATIVE CASE HISTORIES

The case histories selected for more detailed review to illustrate new developments are:

(i) Postal Square, Washington, D.C. - A classic application of conventional pinpiles as an advantageous construction alternate to hand underpinning or large diameter caissons during the redevelopment of an old, massive structure, and which demonstrates the benefits of cyclic loading.

(ii) Brooklyn Queens Expressway, NYC - high capacity pinpiles of more contemporary concept installed as temporary and permanent supports during the renovation of a major overhead freeway.

(iii) Presbyterian Hospital, Pittsburgh, PA - High capacity pinpiles installed in very difficult access conditions, and subsequently partially exposed to operate without lateral restraint.

2.1 Postal Square, Washington, D.C.

Background. The original portion of the massive Old Postal Building, Postal Square, was completed in 1911. A major extension followed in 1931. For many years it served as the main Post Office for Washington, D.C., being located adjacent to the Union Station on Massachusetts Avenue, a few blocks north of the Capitol Building.

The developer (Gerald D. Hines Interests), acting for the Federal Government, planned to remodel the existing structure by adding new office floors in the center court area and constructing mechanical space below the existing lowest basement elevation of +7 m. This meant that existing foundations had to be upgraded and new columns added to support new interior framing. The existing supports are steel and concrete columns on large concrete footings, and 356 mm square caissons end bearing on dense sands.

Originally a very cumbersome underpinning scheme was considered, involving hand dug support, massive spread footings and large diameter caissons, both hand excavated and mechanically drilled. However, the hand work would probably have caused significant undermining of the existing footings, leading to settlement, whereas drilled caisson work would have been inhibited by the very restrictive access, and low headroom. Both techniques would have been unattractively time consuming and costly. The pinpile alternate resolved both concerns.

Site and Ground Conditions. The work was conducted underground in three main areas in the basement of the existing structure:

- B2 Level (EL 1.8 m): large level area with about 3.6 m headroom. Piles reached El -13.5 m.
- B2 Level (El 3.3 m): most restricted area, headroom 2.5 m. Piles reached El -13.5 m.
- B1 Level (El 7 m): Open access with 5-6 m headroom. Piles reached El -10.6 m.

Under the concrete footings and a little fill, the natural soils comprised Recent alluvials, ranging from coarse to fine sands, laterally and vertically variable. Some gravel and mica were found sporadically, together with thin layers of cobbles or stiff clayey silt and silty sand in lower reaches. Typically the sands were dense to very dense. Natural groundwater level was at about El -5'.

Design and Construction. Past experience and standard texts (PTI, 1986) were used to design 390 piles in the B2 levels and

310 piles in the B1 level, each with a nominal working load of 68 tonnes. About 25% of the piles were installed in groups of 4 or 6, through 15 existing B2 (El 1.8) footings comprising 2-4.5 m of concrete. Pile centers were within 500 mm of existing columns.

Totals of 21 new caps were created in B2 (El 1.8), 17 in B2 (El 3.3) and 53 in B1. These featured standard (and several non-standard, specially designed) plan geometries from 1.5 x 1.4 m (3 piles) to 2.3 m square (9 piles). The minimum pile separation was 660 mm center to center, but was typically 760 mm.

Custom built, short mast diesel hydraulic track rigs were used to rotate 178 mm dia. 13 mm wall N80 casing with water flush, to target depth. Type I grout of w/c = 0.45 was injected under excess pressures of 0.6-0.8 Mpa during progressive extraction of the casing for 8 m. The casing was then reinserted 1.5 m into this pressure grouted zone as permanent support. The lowermost 8 m of pile was reinforced by Grade 60, 36 mm dia. rebar in 3 coupled sections.

For those holes through existing footings, a 225 mm down-the-hole hammer was used to penetrate until significant steel was encountered. Thereupon, the hole would be completed with a 200 mm core bit. Load transfer between the casing of the pile and the footing was ensured by the use of a special non-shrink, high strength grout. For the new pile caps, the pinpile casing was extended 100 mm up into the subsequent concrete, the horizontal reinforcing of which was fixed 50 mm above the top of casing.

<u>Testing and Performance</u>. Four special test piles (TP) were installed prior to constructing the production piles (<u>Table 3</u>).

TP1 and 2 were tested cyclically, yielding the analysis provided in <u>Figure 4</u>. TP3 and 4 were also tested incrementally but progressively to maximum load in accordance with ASTM-D1143-81. TP1 clearly failed at the grout-soil interface, the founding horizons being on average finer and less dense than those for the other piles. <u>Figure 4</u> also shows that the elastic compressions of TP1 and 2 were similar at the failure

<u>TABLE 3 -- Test pile data, Postal Square, D.C. (in Imperial Units: as recorded)</u>

	TP1	TP2	TP3	TP4
Area of Operation	B2 (El 1.8)	B2 (El 1.8)	B2 (El 3.31)	B1 (El 7)
Length (ft)	36	51	36	58
Bond (ft)	25	25	25	25
Max. test load (kips)	187.5 (failed)	300	300	300
Elastic extension at max. load (x0.001")	173	461	383	374
Permanent deflection at end of test (x0.001")	313	291	187	113
Total cumulative creep (0.001") during test	96	174	59	61

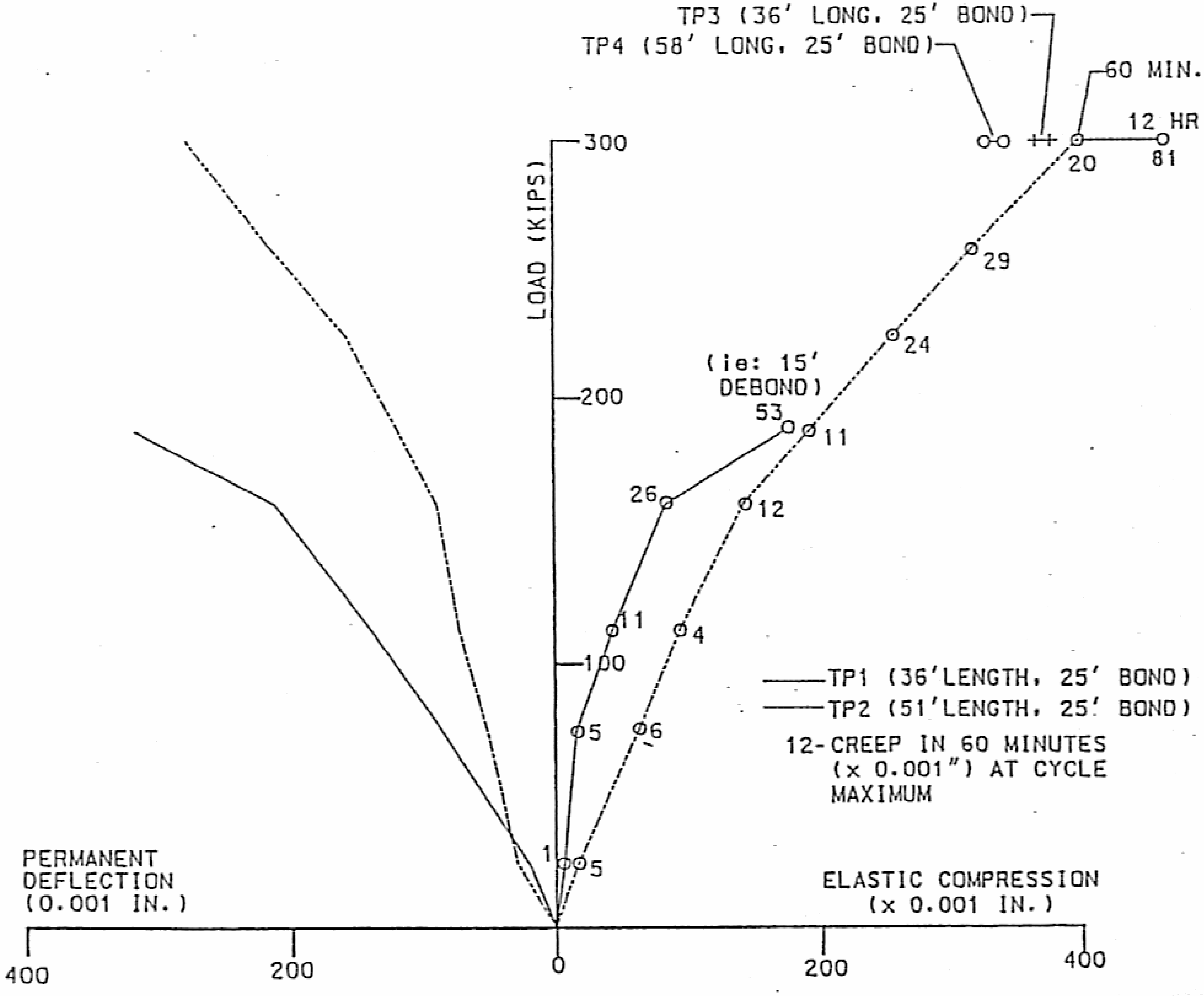

FIGURE 4. Elastic/permanent set performance of TP1, TP2, Postal Square, D.C. (Imperial Units)

load of the former. This shows that load must have been transferred to similar depths in both piles, despite the nominal difference in "free length" upon construction. The elastic performance of TP3 and 4 was likewise similar, supporting the observation.

Table 3 also highlights higher total creep amounts in TP1 and 2: simply a reflection that there were far more creep monitoring periods in the cyclic loading that in the progressive loading. This clearly impacts overall permanent displacement and is an important point to bear in mind when judging pile performance by this criterion.

A separate pullout test was conducted in an existing column footing in the B2 (1.8) level to explore concrete/grout bond. A special element was grouted 1.4 m into a 225 mm hole drilled through the concrete. A special high strength, non shrink grout was used. After repeated cyclic loading to 240 tonnes (79% GUTS, and equivalent to 350% design load), the maximum uplift recorded was 0.13 mm, reducing to 0.03 mm upon destressing. Assuming uniform bond distribution, an average grout-concrete bond of almost 2.4

Mpa had therefore been safely mobilized.

Following installation of the pinpiles, the structural renovation has progressed, and the foundations have performed perfectly.

2.1 Brooklyn-Queens Expressway

Background. The Brooklyn-Queens Expressway exists as a six-lane viaduct between Metropolitan Avenue and Kingsland Avenue in the Borough of Brooklyn. It runs in a north-south direction approximately one mile east of the East River which divides Manhattan from Queens and Brooklyn. This facility is designated as Interstate Route 278 and is a portion of a major artery. It is the only controlled-access expressway to connect the boroughs of Queens and Brooklyn and it provides an expressway link for several counties with the Williamsburg, Manhattan, Brooklyn and Verrazzano Bridges and the Brooklyn Battery Tunnel. This section of expressway was completed in the early 1950's and consists of a series of simply supported spans resting on pile supported bents.

A major improvement program was put into place to replace the deck of the viaduct and to add a new center lane and several new entry-exit ramps. These were needed to correct access, geometric and safety deficiencies which were exacerbated by severe traffic congestion experienced particularly during rush hours.

Due to this viaduct being a portion of a major arterial highway with high traffic volumes, maintaining traffic became a fundamental criterion for project approval. In order to maintain a minimum of two (out of three) lanes of traffic in each direction during construction, a temporary viaduct, adjacent to the existing structure had to be constructed prior to lane closures in each direction to accommodate rehabilitation.

Small diameter (approximately 300 mm) bored piles were specified for the permanent viaduct and ramps, and larger diameter (600 to 900 mm) bored piles were specified for the temporary viaduct. Bored piles were specified for this project due to the otherwise adverse vibration effects that pile driving impact hammers would have on the many adjacent old and sensitive buildings.

This was the first New York State Department of Transportation project where pinpiles were specified to be designed by a prequalified contractor to meet predetermined design capacities. The general contractor was Yonkers Contracting Company, Inc. who sublet the installation of most of the pinpiles to Nicholson.

Site and Ground Conditions. The general foundation conditions at the site were highly variable, but generally consisted of

- 3-5 m of loose to medium compact miscellaneous fill containing silt, sand and gravel with bricks and the like;
- up to 9 m of layers and lenses of loose silt, sand and clay (organic near surface);
- up to 15 m of compact silty sand, occasionally gravelly;
- stiff varved silty clay and clayey silt (Gardiners clay).

Bedrock was not encountered to the maximum explored depth of about 35 m. Generally the compact silty sand and lower reaches of the

Table 4 -- Summary of pinpile requirements, BQE, NY

CONSTRUCTION STAGE	NUMBER OF NEW PILE CAPS	PILE DIAMETER	DESIGN CAPACITY (TONNES)
1 (eastbound permanent viaduct)	Approx. 30	Approx. 300 mm	45, 73, 90
2 (temporary viaduct	Approx. 60	600 - 900 mm	Variable: 55 to 132
5 (westbound permanent viaduct)	Approx. 30	Approx. 300 mm	46, 72, 90 (to be installed)

lenses of silt, sand and clay were recognized as being adequate load bearing materials commencing at a depth of about 15 m below the existing ground surface. The piezo-metric level was encountered between 3 and 5 m below the existing ground surface.

There were some access and headroom restraints particularly where drilling had to be performed under the existing viaduct (about 5 m headroom). In addition, construction had to accommodate traffic control, protection of buried and overhead utilities and noise and vibration impact mitigation.

Design. Approximately 120 new pile caps, each with between 2 and 10 piles per cap were proposed as shown in Table 4 in different stages to accommodate maintenance and protection of traffic.

All piles were to be designed without batter. The small diameter piles were specified to be designed as friction piles by a prequalified contractor to meet the design capacities. The prequalification consisted of requiring the contractor performing the work to submit proof of: 1) two projects on which he had successfully designed and installed similar bored piles or tiebacks, using non-displacement methods under similar site conditions; and 2) the foreman having supervised the successful installation of the same on at least two projects in the past two years. The specifications indicated that the grout mix, steel casing and/or reinforcement had to meet specified minimum requirements and included general provisions concerning shop drawing submittals, drilling, casing removal, post grouting, and construction tolerances. The contractor's proposal to found the temporary viaduct on pinpiles in lieu of the larger diameter piles was approved by the State with the provision that the piles be battered where permitted by right of way and utility conditions and the pile caps be tied to the permanent viaduct in the direction where piles could not be battered to provide the necessary lateral restraint.

The specification called for completing several successful static pile load tests as a basis for pile acceptance (Table 5).

Two different pile designs and general installation procedures were submitted by Yonkers and Nicholson respectively and were

TABLE 5 -- Tests required at BQE, NY

	On non-production piles prior to installation of production piles	On production piles
Permanent Viaduct	4	Bents 19, 26, 31, 37, 44 and at 3% of remaining at locations designated by the Engineer.
Temporary Viaduct	2	Bents 27, 37, 50, 69, and at 1% of remaining at locations designated by the Engineer.

TABLE 6 -- Details of Yonkers and Nicholson pinpiles, BQE, NY

YONKERS	NICHOLSON
230 mm O.D. casing is advanced using duplex drilling with air flush. Klemm drill conterrotates the casing and 101 mm inner drill rods. (Double Head Duplex: Bruce (1989b).	178 mm O.D., 13 mm wall grade N 80 casing is drilled in using water flush.
Install 2.5 or 46 mm grade 60 rebar after drill rods are removed.	Place grout tremie to bottom and place 34 Mpa neat cement grout.
Place grout hose to bottom and place 34 Mpa neat cement grout.	Pressurize grout to about 0.6 Mpa as casing is withdrawn 4 m.
Pressurize grout with air to 0.7 Mpa as casing is withdrawn about 10 m.	Readvance casing to bottom to serve as reinforcement.
Inject the top section of pile under moderate pressure as casing is removed.	

approved by the state contingent upon obtaining successful pile capacities from the static pile load tests (Table 6).

Construction. Piles were installed as foreseen, and subsequent testing (below) showed that, when founded in silty sands, the 90 tonne design capacity was readily attained. However, when the bond zone predominantly consisted of looser deposits of silt, sand and clay, the guaranteed minimum design load was estimated as 55 tonnes. This inability to reach high loads reflected the fact that the soils would not naturally "seal" around the drill casing, so preventing the application of the target grouting pressures. A test with postgrouting techniques did raise grout/soil bond capacities to a level capable of providing the original design load. However, the contractor, given the overall site and project restraints, proposed instead to derate the pile design capacity in these areas to 55 tonnes and to install more piles (at no extra cost to the State). This proposal was found acceptable, and so pile depths typically varied from 15 - 18 m although in one area they were taken deeper so as to avoid extra loading on existing subway tunnels.

All piles were loaded cyclically, incrementally in order to provide data on both elastic and permanent displacements (Bruce 1991). This was done basically in conformance with Soil Control Procedure SCP-4/77. To date a total of 18 tests have been performed (9 non-production and 9 production piles) as summarized in Table 7.

Regarding permanent movements, Figures 5 and 6 summarize the performance of piles founded in the silty sands, and clayey silts respectively. Creep data reflected the same divergence in performance.

As shown in Table 7, the pile at Bent 27C was postgrouted after initial testing (Figure 7). One may compare the original maximum achieved load of 160 tonnes (permanent deflection 18 mm), to the subsequent, easily attained load of 182 tonnes (5 mm permanent deflection). After postgrouting, the creep was 0.3 mm during the last 4 hours at 182 tonnes.

One lateral loading test to 9 tonnes gave a total deflection of 19 mm, and a permanent displacement of 3 mm.

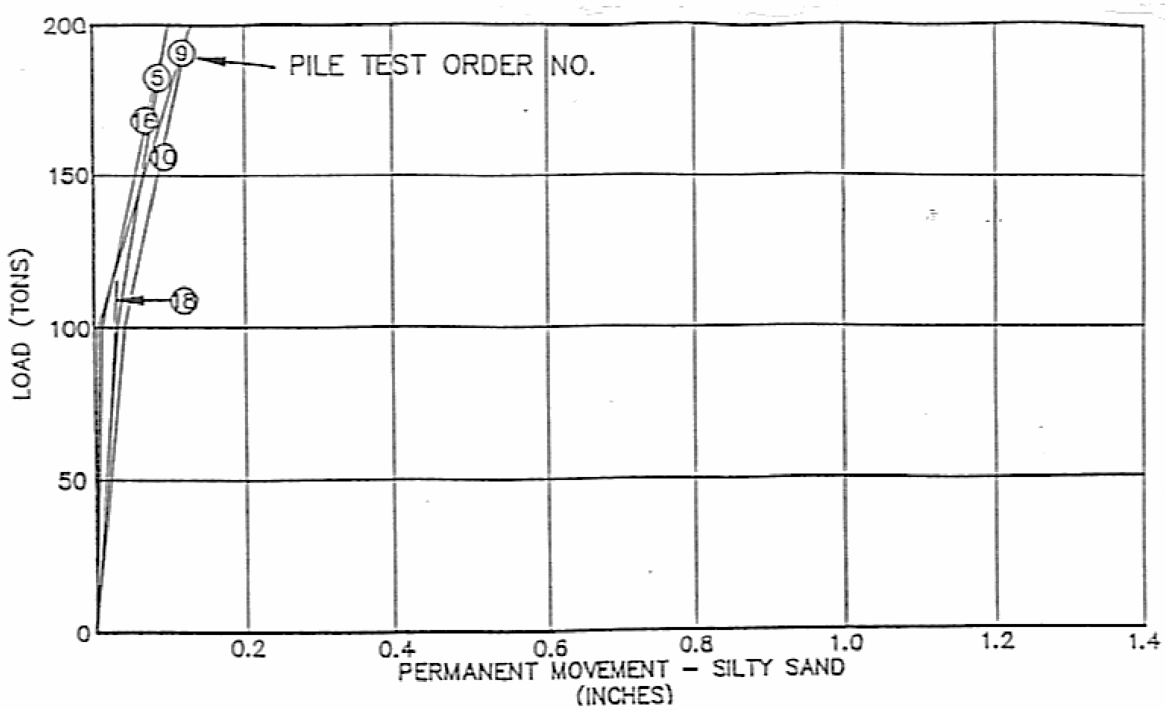

FIGURE 5. Permanent displacements of test piles founded mainly in silty sand. BQE, NY (Imperial Units)

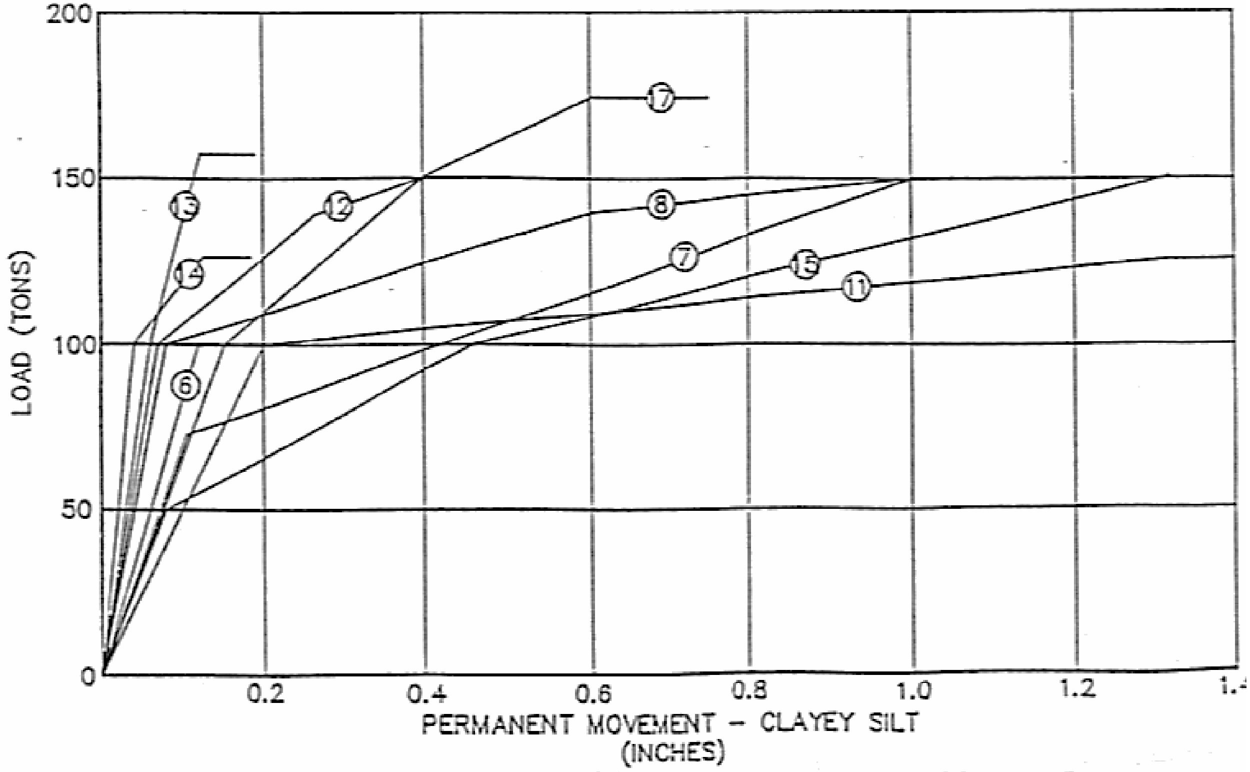

FIGURE 6. Permanent displacements of test piles founded mainly in silts. BQE, NY (Imperial Units)

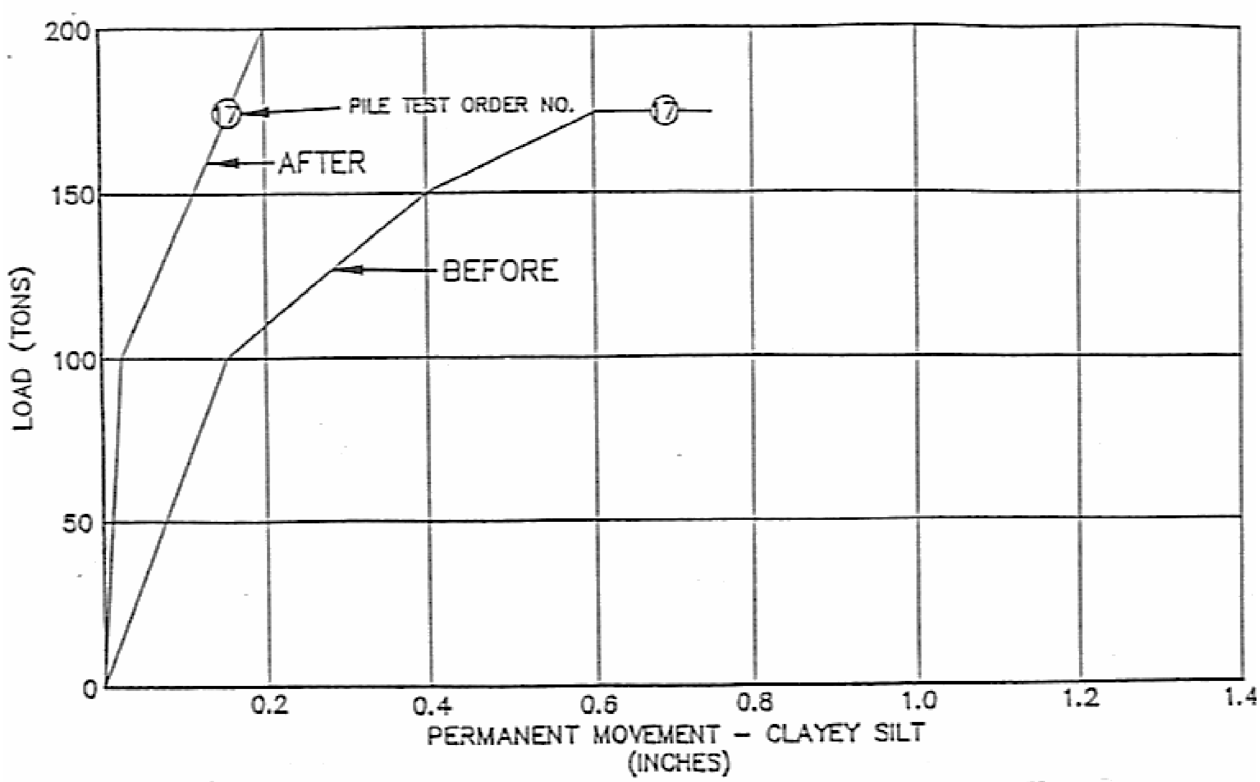

FIGURE 7. Performance of test pile at Bent 27C, in clayey silts, before and after postgrouting. BQE, NY (Imperial Units)

LOCATION	BETWEEN BENTS 35 & 36	BETWEEN BENTS 38 & 39	BETWEEN BENTS 38 & 39	BETWEEN BENTS 30 & 31	BETWEEN BENTS 15 & 16	BENT 34	BENT 31	BETWEEN BENTS 12 & 13	BETWEEN BENTS 12 & 13	BENT 16	BENT 26	BENT 11B-C	BENT 26C	BENT 28C	BETWEEN BENTS 9 & 10	NEAR BENT 1	BENT 27C	BENT 82
LOAD TEST NO.	TP-1	TP-2	TP-3	TP-4	TP-5	P-1	P-2	1	2	3	4	7	5	6	–	–	8	T-1
ORDER NO.	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
PERM. OR TEMP.	PERM.	PERM.	PERM.	PERM.	PERM.	PERM.	PERM.	PERM.	PERM.	PERM.	PERM.	PERM.	PERM.	PERM.	PERM.	PERM.	PERM.	TEMP.
PROD. OR NON-PROD.	NON.	NON.	NON.	NON.	NON.	PROD.	PROD.	NON.	NON.	PROD.	PROD.	PROD.	PROD.	PROD.	NON.	NON.	PROD.	PROD.
AGE AT TEST (DAYS)	37	13	N/A	14	29	N/A	N/A	5	6	5	7	19	7	7	N/A	N/A	7 3 POST GRT.	8
NOMINAL DIA. (INCHES)	9.05	9.05	9.05	9.05	9.05	9.05	9.05	7	7	7	7	9.05	7	7	9.05	9.05	7	7
NOMINAL BOND LENGTH (FEET)	35+/-	35+/-	35+/-	35+/-	35+/-	35+/-	35+/-	15	18	7	6	20	20.5	21	35+/-	35+/-	15	30
TOTAL LENGTH (FEET)	57	59	N/A	65	78	N/A	N/A	40.5	50	50.5	50	50.5	62	66.5	61	61	50	50
ORIGINAL DESG. LOAD (TONS)	50	100	100	100	100	50	75	100	100	100	100	100	100	100	100	100	60	60
MAX. TEST LOAD (TONS)	186	146	125	175	275	100	150	165	200	200	125	150	160	125	175	200	175 200 POST GRT.	120
TOT. DEFLEC @ MAX.LOAD (INCHES)	N/A	N/A	0.51	0.86	0.36 @200T	0.29	1.2	1.7	0.51	0.42	1.85	0.77	0.43	0.38	2.7	0.58	1.24 0.72 POST GRT.	0:18
PERM. DISPL AFTER MAX LOAD (INCHES)	N/A	N/A	FAILED	FAILED	0.14 @200T	0.10	FAILED	FAILED	0.13	0.13	FAILED	0.44	0.12	0.13	2.2	0.10	0.74 0.20 POST GRT.	0.03

TABLE 7 -- Summary of test pile data, BQE, NY (Imperial Units

2.3 Presbyterian University Hospital, Pittsburgh, PA

Background. The Presbyterian University Hospital complex already occupies two extremely congested city blocks, and so when the need for more facilities became apparent, the decision was made to vertically extend and laterally link several existing operational structures. Overall, 160,000 square metres of new facilities are being built in four major additions.

This highly delicate operation, conducted within a fully functional facility, has necessitated some equally complex and innovative foundation engineering solutions involving excavation support and structural underpinning. One of the most dramatic operations has been associated with the completion of a new Magnetic Resonance Imaging Center. The construction of a new elevator pit called for a 9 m deep excavation directly underneath three exterior column footings of the adjacent 13-story hospital structure. The pit, 18 x 10 m in plan, was further bounded on two sides by five additional footings, and so these sides required anchored lateral support.

Historically, the support of columns in such circumstances has been achieved by conventional underpinning pits and needle beams. However, in this instance, the difficult access conditions, and the specified requirement to limit downwards movement of the columns to less than 3 mm demanded a special solution, featuring high capacity pinpiles in rock.

Site and Ground Conditions. As noted, the access was very restricted laterally and vertically (as low as 3.6 m), and the work had to be conducted within the confines of a fully operational medical facility. The piles were installed through one metre of existing nearby reinforced concrete footings cast directly on fractured, fissile medium hard-hard siltstone, occasionally calcareous or limey.

Design and Construction. At each existing footing, six pinpiles (4 working, plus 2 redundant) were installed in 216 mm holes drilled vertically by rotary percussive methods and air flush to the target depth (13 m below the footing). Each had a design working load of 114 tonnes. The reinforcing element consisted of a 178 mm dia., 13 mm wall N-80 casing placed full depth in each hole which was then tremied full of neat cement grout of w/c = 0.45. The upper 7 m of each pipe was greased on the outside to debond it from the surrounding grout in that region and so permit load transfer into the 6 m long bond zone. The suitability and security of this design had been proved in the earlier test program, described below.

A structural steel jacking frame was then erected over the top of the piles and fastened to the existing steel column. Each of the steel columns - supporting an occupied hospital building - was then sequentially lifted off its existing spread footing by a distance of 1.3 - 3 mm. This effectively preloaded the piles to prevent any later settlement of the building, and transferred the column loading into the bedrock, but 7 m below. Excavation then proceeded, supported laterally by beams, shotcrete lagging and prestressed rock anchors. As the excavation deepened, cross frames were welded to the pinpiles to limit the unbraced lengths of these piles, now exposed as steel columns.

Testing and Performance. By the end of excavation, the foundations of the existing structure could be seen resting on the pinpile groups, 6.6 m off the bottom of the excavation. During and after excavation, absolutely no movement of the structure could be measured.

One of the most common problems foreseen for pinpiles (Bruce and Yeung, 1983) is the potential for buckling or bending, as an inferred consequence of their high slenderness ratio. This unique project - featuring pinpiles with no surrounding ground to offer any lateral restraint - is proof irrefutable that correctly designed and constructed pinpiles can operate with surprising efficiency not only in the axial sense.

Clearly, testing of production piles was not possible in this instance, and so a full scale test pile was installed beforehand. Using identical construction methods, a pinpile with 6 m bond was formed in the same geological stratum. The total length was 15 m, including, therefore, 9 m of debonded "free length". Reaction to the test load was provided by a pair of prestressed rock anchors. The casing was preassembled in the workshop and consisted of 5 separate lengths, hand tightened together. Two "telltales" were incorporated - one each at the top and bottom of the bond zone. A thick, soft wood plug was attached to the bottom of the reinforcing pipe to eliminate any possible end bearing contribution and to so allow only side shear to be mobilized. As part of the contract requirement, the pile was then tested to twice design working load (228 tonnes), according to ASTM-D1143-81 (modified to allow cycles at 25%, 50%, and 75%). Results are summarized in Table 8. At 114 tonnes, the elastic compression of 0.6 mm was exactly that predicted, while the permanent displacement of 1.3 mm was proved by the telltales to be due to some inelastic compression of the steel casing itself. While loading from 182 to 193 tonnes, a "bump" was recorded and the load immediately dropped to 136 tonnes. Load was then increased to 227 tonnes when a further "bump" occurred. However, when the data from the cyclic loading and the telltales were analyzed it became clear that:

(i) the pile elastic deflection at 227 tonnes was exactly as predicted, and

TABLE 8 -- Summary of test pile performance, Presbyterian University Hospital, Pittsburgh. (All movements in thousandths of an inch - Imperial Units).

Load cycle maximum (kips)	Total butt movement at maximum (A)	Apparent permanent butt movement at subsequent zero (B)	i.e. Elastic deflection at maximum (A) - (B)	Bottom telltale movement (relative to butt)
125	127	42	85	37
250	279	52	227	44
375	448	77	371	63
300	663	329	334	295
500	1020	517	503	471

(ii) the apparently large permanent movement (Table 8) was due to an irreversible "one off" shortening of the steel pipe. The assembly records of the pile were then reviewed. It transpired that there had been several "unshouldered" hand tightened joints between adjacent casing sections. It was suspected that each joint was unshouldered about 3 - 6 mm. Thus, the sudden 12 mm permanent compression of the pile material was readily explained, and when subtracted from the permanent set of 13 mm, gave a true movement of the pile tip into the rock of 1 mm at 227 tonnes - an outstanding performance. There was negligible creep at all load increments.

Thereafter, the pile was tested to a maximum load of 307 tonnes before it became clear that material failure of the steel casing under the jack was occurring. At this load, the steel had compressed 78 mm (from telltales), compared with the measured butt permanent displacement of 82 mm. Thus, at 307 tonnes, a true permanent movement of the pile of 4 mm had been recorded, while analysis proved the perfect elastic performance of the pile with a calculated debonded length barely one metre into the bond zone.

This project was therefore highly significant from several viewpoints:
(i) the excellent lateral and vertical performance of pinpiles was proved again;
(ii) the value of telltales in aiding understanding of internal pile performance was demonstrated, and
(iii) the warming fact that the boundaries of pinpile design are now those of the constituent materials - i.e., independent of the surrounding ground properties - was massively underlined.

3. FINAL REMARKS

These recent case histories clearly underline the advances being made in pinpile technology in the United States. One can cite the unusually high load capacities, the use of preloading and postgrouting, the quality of the testing, and the growing understanding of lateral loading behavior. In favorable geological conditions, the use of contemporary drilling and grouting techniques has allowed so much grout/soil bond potential to be exploited, that the limit to load holding capacity is the internal composition of the pile itself, i.e., the grout and the steel reinforcement. Tests currently underway in Port Vancouver, WA, and Olin, AL, are permitting optimization of internal pile structure to the extent that pin piles approaching 400 tonnes test load in dense sands are rapidly becoming feasible. While such load capacities may be only rarely needed, they are illustrative of the pace at which pinpile performance is being systematically enhanced.

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