Papers

Aspects of minipiling practice in the United States

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Minipiling has gained acceptance in the US during the past decade. Bruce describes a variety of tasks completed by his firm using the technique.

Introduction

To date the bulk of the data published on minipile practice has originated in Western Europe and the Far East. For example, in the former category several papers published in this journal (Koreck 1978)[1] (Weltman 1981)[2] and (Attwood 1987)[3] largely describe the British approach while Lizzi (1978)[4], (1982)[5] and Mascardi (1970)[6], (1982)[7] have long recounted the classic Italian procedures. Developments in Germany (Herbst 1982)[8], France (Gouvenot 1975)[9], and Holland (Doornbos 1987)[10] have also been frequently described. In the Far East, papers by Mitchell (1985)[11] and Bruce and Yeung (1983)[12] illustrate applications in Malaysia and Hong Kong respectively.

The increasing popularity of minipiling as a routine method of underpinning reflects the trend of the civil engineering industry in those regions: the emphasis is on infrastructure development, redevelopment and upgrading in areas of high population density. Such activities often lead directly or indirectly to the need for the underpinning of existing structures. In urban conditions where access, programme, soil, structural and environmental considerations often conspire to complicate and limit construction options, minipiling has proved to be an excellent choice, with almost 40 years of successful case histories to cite.

In the United States, these particular ramifications of undertaking complex ground engineering works in existing urban and industrial environments have impacted rather later (Bruce 1988)[13]. It is only within the last decade or so that many of the older metropolitan areas have invested in mass transit systems involving bored and open cut tunnelling, and several major new sewerage and waste water schemes are now being constructed similarly. At the same time, the final links of the Interstate highway system are being built or expanded through the cities they connect.

As always, necessity conceives invention, and this change in construction emphasis – affecting the industrial and dam engineering sectors equally – is reflected in the recently perceived American requirement for the new European technologies (Nicholson 1987)[14]. The contraceptive – to continue the maternal metaphor – has been the very restrictive and litigious contractual atmosphere which has most definitely not encouraged innovation and has frequently penalised imperfection.

Pic. 1. Typical minipile installation conditions. Boylston St. Boston, MA.

Fortunately, engineering demands are proving more potent than legal interruptions in the field of minipiling, and in the last 10 years there has been a major expansion in the popularity of the technique in the United States. At the time of writing it would seem that the volume of minipiling conducted in cities such as Boston and New York is at least on a par with that executed in London or Paris, while the potential is inestimable.

Pic. 2. Drilling at relocated pile positions to accommodate special access restraints, Boylston St. Boston, MA.

As an introduction, Table 1 summarises the details of some contracts executed over the last few years by the author's company alone. These examples exclude the frequent cases where minipiles (or 'pinpiles' in local terminology) have been used as reaction elements in the course of large rock anchor tests, or where they have been installed as in situ reinforcement for slope stability (Bruce and Jewell 1986-87)[15] or where they have been used as simple pins to stabilise the toes of sheet pile walls. This table illustrates several points familiar to European practitioners:

* The wide range in the scale of individual projects.
* The range in design working loads.
* The excellent load/settlement performance.
* The relatively narrow range in dimensions.
* The common applications in restricted headroom conditions within existing structures.
* The frequent use of a full length permanent steel tube passing down into the load transfer zone.
* Their installation in virtually all soil conditions.

In terms of value, the largest project yet conducted in the United States appears to have been the reconstruction of the Hynes Auditorium in Boston, MA (almost $7M in 1986 involving about 60 000 lineal feet of 13½" diameter piles† of capacities up to 250 tons) (ENR 1986)[16]. Looking to the future, however, a project is underway in New York City where the value of the

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†All units are Imperial, in deference to US practice. In addition, 1 ton is 2000 pounds, and 1ksi is one thousand pounds per square inch. Rebar sizes are readily calculated: the number times ⅛" gives the diameter ie a number 8 rebar is 1" diameter.

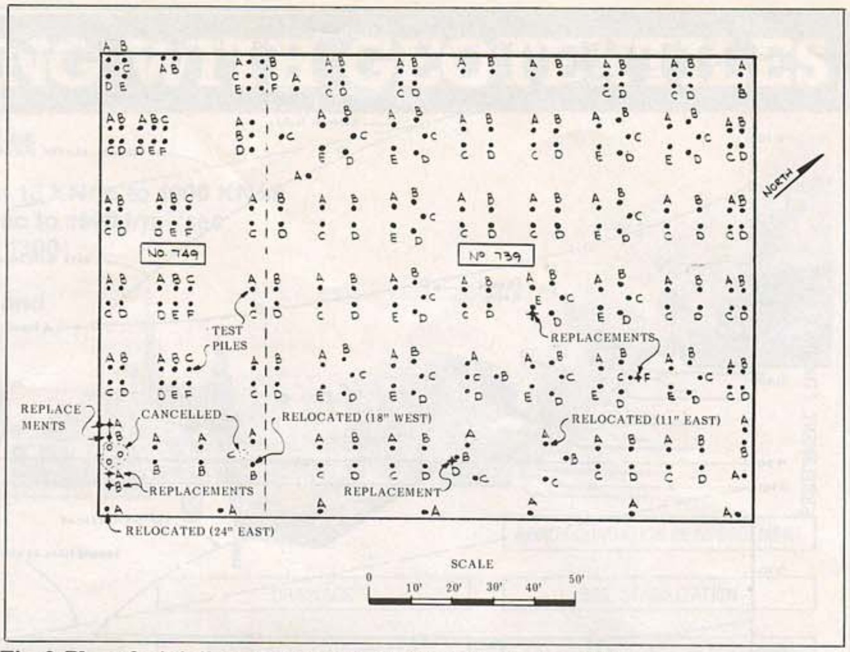


Fig. 2. Plan of minipile arrangement, Boylston St., Boston, MA.

minipiling alone will be substantially in excess of even that figure. Most significantly, that particular project will feature the use of the postgrouted pile system similar to that described by Jones and Turner (1981)[17] and Rodio (1984)[18]. That will be, to the author's knowledge, the first such application in the US, although postgrouted anchorage systems are becoming popular in the east and south of the country. Design working loads of 100 tons (with testing to 200 tons) are to be provided by piles of 12″ nominal diameter founded in 30′ of dense, fine and medium sands.

The case histories which follow have been selected to illustrate approaches to design and construction, and to provide additional data on minipile performance for general information. Comparative recent British practice is described by Bruce et al (1985)[19] and Attwood (1987)[3], who also give general background on minipile characteristics, applications, design and construction, not otherwise discussed in this review.

Details of selected case histories.

Details from six significant case histories are provided in this section. The first three (Boylston Street, Hynes Auditorium and Coney Island) are classic minipile applications within which the typical American approach to design, construction and testing can be illustrated. The fourth (Brookgreen Gardens) is an excellent demonstration of the advantages of minipiling as a flexible, compatible construction technique in adverse conditions. the other two (Warwick and Warren County) describe recent developments in the use of preloading, and very high capacities, respectively. To aid comparison, each case history is presented in the same format.

Boylston Street, Boston, MA

Background

The properties at 739-749 Boylston Street in the Back Bay area of Boston, Massachusetts, were completed in the 'Chicago style' in 1906. These derelict commercial buildings, six and three storeys high, were acquired for redeveloping and refurbishing: the former, for example, will have retail space on the basement and first floors, office space to the eighth floor and a mechanical penthouse level above.

The structure was founded originally on pile caps bearing on timber piles. To accommodate the increased loadings from the new construction, additional support was required under enlarged pile caps (Fig. 1).

The Engineer foresaw piles of working loads 20 tons (compression) and 6 tons (tension), but accepted Nicholson's alternative design offering a cased pile with working loads of 40 tons and 12 tons, respectively.

Piling had to be executed from within the partially demolished basement of the structure (approximately floor elevation +8′) about 10′ below existing sidewalk elevation (Pic. 1) giving a minimum headroom of 8′.

Site and ground conditions

Access was awkward and restricted, and the position of several piles had to be adjusted slightly to accommodate particular site conditions (Pic. 2).

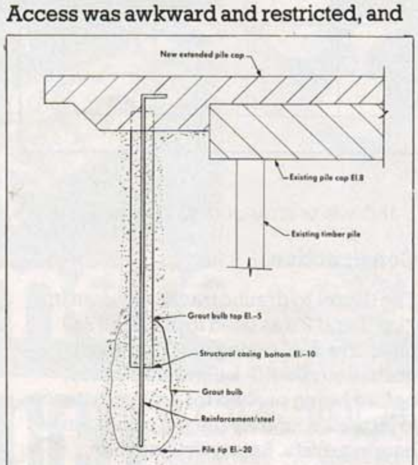


Fig. 1. Minipile detail. Boylston St., Boston, MA.

The fill consisted of saturated loose grey/brown fine sand and silt, and overlaid soft grey organic silt with traces of shells, sand and gravel. The founding layer occurred at about −4′ and was 18′ to 24′ thick throughout the site. It comprised medium dense/dense fine medium sand with a trace of silt. Pile lengths were maintained within this horizon so as not to perforate the underlying Boston Blue Clay.

Design

Piles were designed on the basis of an ultimate load 2.3 times design working load (ie 92 tons in compression, 27 tons in tension).

The length of the load transfer zone was designed on the basis of analogous soil anchor experience and assumed $\varnothing = 35°$ for the sand, and a bulb diameter of 7½″ using a grout pressure of 60 psi in these soils.

Ultimate soil/grout bond ($\tilde{\tau}$ ult) was estimated empirically (Littlejohn 1980)[20] from

$$\tilde{\tau}\text{ ult} = \text{grout pressure} \times \tan \varnothing = 60 \times 0.7 = 42 \text{ psi}$$

Thus for an ultimate load of 92 tons, the required load transfer length (L) is

$$L - \frac{Load}{\pi d}\,\tilde{\tau}\text{ ult}$$

where d is the bulb diameter

$$\text{ie } L - \frac{92 \times 2000}{\pi \times 7.5 \times 42} = 186'' \text{ ie } 15'\ 6''$$

Further routine calculations using the provisions of the Massachusetts Building Code (as described in the design for Hynes Auditorium, following) demonstrated that:

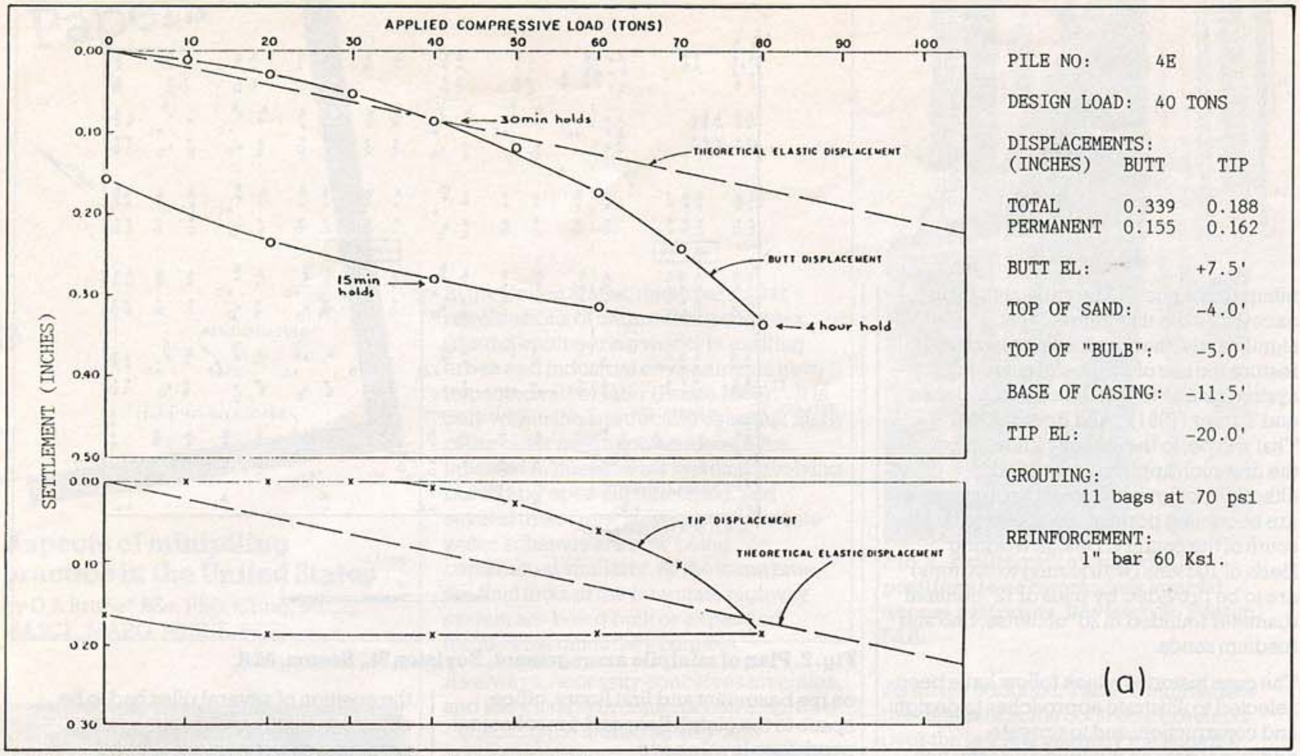


Fig. 3a. Performance of conventional minipile, Boylston St.

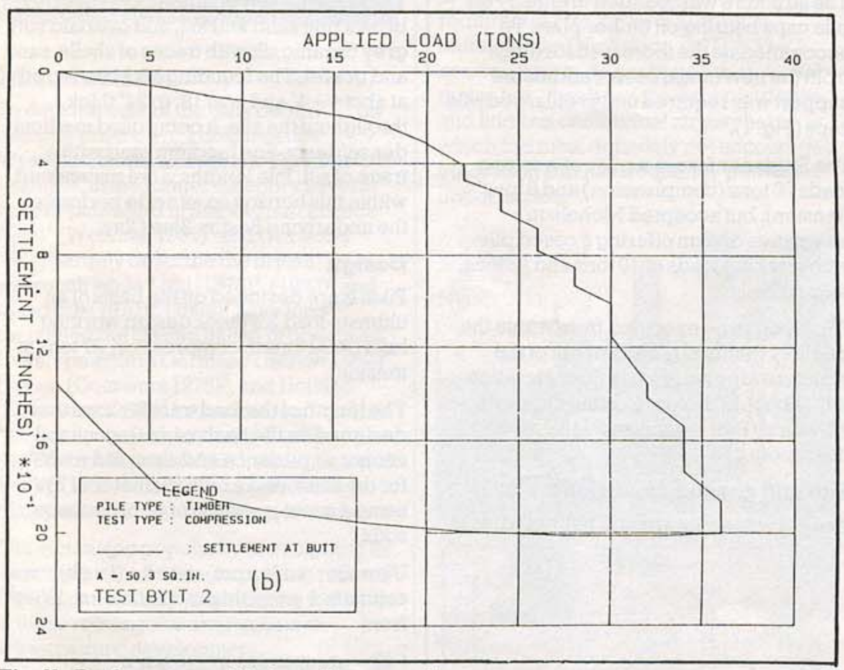


Fig. 3b. Performance of timber pile, same site.

* The use of 5½" casing of 0.362" wall thickness, and fy (minimum specified yield stress) = 55 ksi as the major load bearing element, was safe.
* The anticipated pile settlement at working load was acceptable.
* The compressive strengths generated in the grout of the bond zone were acceptable.
* The use of an internal 1" diameter 60 ksi rebar would adequately transfer loads in the founding horizon.

The individual piles were as shown in Fig. 1 and were arranged as in Fig. 2.

Construction

The diesel hydraulic trackrig shown in Pics. 1 and 2 was used to install all 260 piles. The 5½" casing was first water flushed to about 8' below the surface, before being pushed for a short distance to locate accurately the top of the dense bearing strata. Rotary drilling then resumed in the sand to full depth. Neat Type I grout of water cement ratio (w) about 0.50 was placed by tremie, followed by the rebar. Pressure grouting of the sand was carried out to a maximum of 60 psi during extraction of the casing, for the 15' to 16' of bond zone. The casing was then pushed back down about 5' into this pressure grouted zone and left in place.

Grout takes generally ranged from 2.5 to 3.5 times nominal hole volume, confirming that the enhanced effective diameter of the bond zone had been achieved. Grout cubes at 14 days gave unconfined crushing strengths of over 6000 psi.

During drilling, wood piles or granite blocks in the fill were occasionally encountered but were accommodated by relocation or perseverance. Overall, four piles had to be replaced due to constructional problems, while the instruction of an additional two piles lifted the contract total to 262.

Testing and performance

Prior to the production piling programme, compressive and tensile load testing on two typical piles was conducted.

Each pile was constructed as described above, except for the addition of a 'tell tale' anchored near the tip and the placing of an outer steel liner around the 5½" casing above the bond zone to prevent any load transfer in the upper soils. Reaction for each test pile was provided by adjacent ground anchors, and the tests were executed in accordance with the recently proposed modifications to the Massachusetts State Building Code [21] and ASTM D1143[22]. The data are summarised in Table 2, while the performance of TP2 (in compression) is shown in Fig. 3

together with that of a timber pile, for comparison.

It was noteworthy that the elastic (recoverable) settlement at 80 tons was about half the total deflection, while no indication of pile or soil failure was evident from the butt or tip displacement curves. Furthermore, the net butt settlements were well below recommended Building Code criteria for maximum net settlements. The performance in tension was equally satisfactory.

Most of the major structural rebuilding work was completed in the eight month period following completion of the pin piles. Readings were taken regularly of the pile cap deflections at 16 locations. The range of cap settlements during construction was 0.06" to 0.24" (Average 0.16") and entirely consistent with the test data of Table 2 (Total settlements of 0.34" to 0.44" at twice working load, without the benefit of existing timber piles).

	Butt (inches) TP-1	Butt (inches) TP-2	Tip (inches) TP-1	Tip (inches) TP-2
Compression test (to 80t)				
Gross settlement	0.44	0.34	0.31	0.19
Net settlement (permanent)	0.25	0.16	0.25	0.16
Tension test (to 24t)				
Gross heave	0.24	0.14	0.17	0.06
Net heave (permanent)	0.16	0.09	0.15	0.06

Table 2. Summary of test data on test piles (TP) 1 and 2. Boylston St. Boston, MA.

Hynes Auditorium, Boston

Background

In 1985 the foundation works conducted by Spencer, White & Prentis/Bauer Corporation of America at the Hynes Auditorium in Boston represented by far the largest minipile job executed in the United States. The following description is drawn from Johnson and Schoenwolf (1987)[23], of Haler and Aldrich Inc., the geotechnical engineer on the project.

Hynes Convention Centre was constructed from 1985 to 1987 by renovating and expanding the existing Hynes Auditorium. One floor had to be added above the existing two storey structure and a three storey addition added to the north and east (Fig. 4).

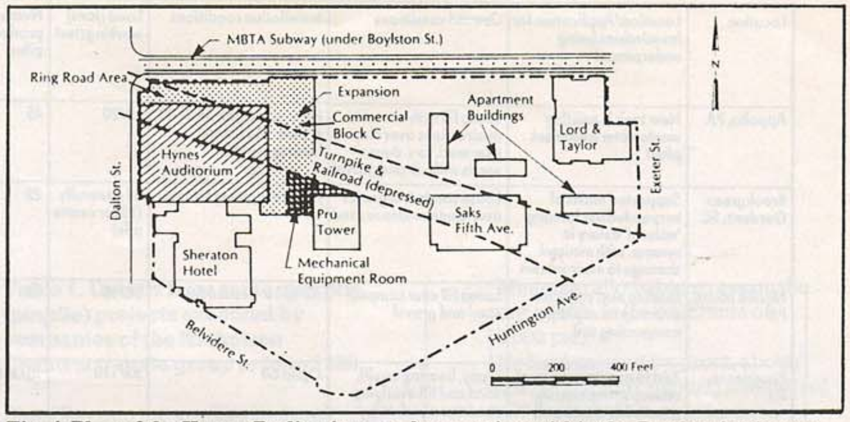


Fig. 4. Plan of the Hynes Auditorium, and expansion within the Prudential Center complex, Boston, MA (Johnson and Schoenwolf, 1987).

For the renovation and expansion, it was judged that the existing pile foundations would be adequate. New piling was needed for the additions. For the north section, conventional driven H piles could be installed. On the other hand 'many site and project constraints' dictated the use of an alternative form of piling for the east section. These factors included

1 Driven piles could possibly have disturbed the clay and caused settlement to adjacent soil bearing foundations (eg for the Prudential Center garage).
2 Restraints were imposed by the position of new columns relative to existing columns (eg the garage south of the turnpike).
3 The need to install piles in the median strips of the turnpike and railroad without interfering with traffic and the need to install them from the floor level above the mechanical equipment room of the Prudential Center.
4 Limited headroom – over 50% of the piles had to be installed from areas of 10' to 14' overhead clearance.
5 A minimum capacity of 175 tons per pile was found to be structurally most efficient.

Site and geology

The intensive site investigation confirmed the typical Boston Back Bay sequence:

Fill – 26' to 35' thick.
Organic silt – 2' to 3' thick (often missing).
Very dense coarse, fine sand plus some gravel and silt – 12' to 16' thick.
Very stiff to very soft silty marine clay – 110' to 142' thick.
Very dense glacial fill – up to 13' thick.
Bedrock (Cambridge argillite, the upper 2' to 9' of which was moderately to completely weathered).

The headroom conditions were restrictive, and together with the need to drill from existing floor slabs, the size and weight of the drilling equipment was furthern circumscribed.

Design

The anticipated loading conditions required the installation of 333 atypically high capacity (up to 250 tons) piles to depths of 150' to 170' through the Boston

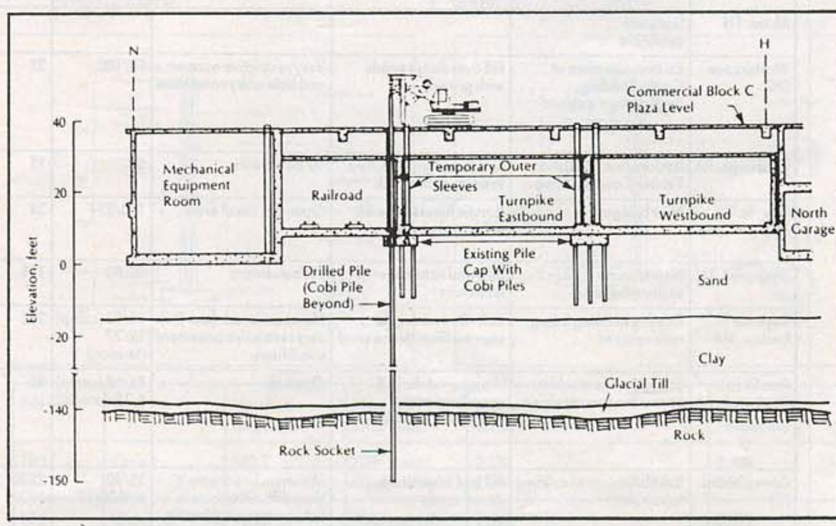


Fig. 5. Section through the turnpike and railroad right of way, and the mechanical equipment room, showing minipile installation through 16" diameter temporary outer liners. Hynes Auditorium (Johnson and Schoenwolf, 1987).

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 Reinforcement & casing
Appollo, 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′
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′
Neville 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′
Providence, R.I.	Test to assess viability of underpinning existing granite block seawall	Quay, bearing on silt, sand and till overlying sandstone bedrock	Open air	55/110	1 (Test)	65	65	6	5″ casing for 57′
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
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′
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′
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′
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
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′
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′
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
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′
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
Appollo, 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′
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′
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
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
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
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
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)	262	7070	27	5½	#8 rebar full length 5½″ casing in upper 19′
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
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
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.

data Grouting	Test performance/special notes
Type I w = 0.5 Maximum press 100 psi	Test data on 2 piles: Total displacement at 20 tons – 0.049", 0.077" Permanent displacement after – 0.008", 0.022" resp.
Type I w = 0.5 Maximum press. 120 psi	Award winning solution to unique set of problems
Type I w = 0.5 Maximum press. 00 psi	Test data on 1 pile: Total displacement at 60 tons – 0.078" Permanent displacement after – 0.01" (Allowable 0.60")
ype I w = 0.45 Gravity fill	Test data on the one pile. Total displacement at 110 tons – 0.70" Permanent displacement after – 0.03"
Type II w = 0.5 Maximum press. 100 psi	Test data on 1 pile: Total displacement at 20 tons – 0.055" Permanent displacement after – 0.005"
Type I w = 0.45 Maximum press. 120 psi	Test data on 2 piles: Total displacement at 55 tons – 0.188" and 0.249" Permanent displacement after – 0.002" and 0.005" resp.
Type II w = 0.45 Maximum press. 100 psi	Test data on 1 pile: Total displacement at 100 tons – 0.312" Permanent displacement after – 0.008"
Type I w = 0.5 Maximum press. 80 psi	Piling part of major overall structural repair.
Type II w = 0.45 Maximum press. 40 psi	
Type III w = 0.45 Gravity pressure	—
Type I w = 0.5 Maximum press. 60 psi	Piles installed in conjunction with subhorizontal soil nails for excavation stability.
Type I w = 0.45 Gravity pressure	—
Type I w = 0.45 Maximum press. 120 psi	Test data on 1 pile: Total displacement at 100 tons – 0.2" Permanent displacement after – 0.02"
Type I w = 0.45 Gravity pressure	—
Type III w = 0.45 Maximum press. 150 psi	—
Type I w = 0.45 Maximum press. 50 psi	—
Type I w = 0.5 Maximum press. 140 psi	1 Piles combined with subhorizontal soil nails to stabilize excavation adjacent to structure. 2 Data on Test Pile 2: Total displacement at 100 tons – 0.653" Permanent displacement after – 0.078"
Type I w = 0.45 Gravity pressure	—
Type III w = 0.5 Maximum press. 50 psi	Test data on 1 pile: Total displacement at 205 tons – 0.40" Permanent displacement after – 0.07"
ype I w = 0.45 Gravity pressure	No measureable permanent displacement after testing to 80 tons.
ype II w = 0.5 Maximum press. 50 psi	Test data on 2 piles: Total displacement at 92 tons – 0.44" and 0.34" Permanent displacement after – 0.25" and 0.16"
ype I w = 0.45 Gravity pressure	1 Piles subjected to vertical, lateral and moment testing. 2 Compression test data on 6 piles: Total displacement at 68 tons – 0.059 to 0.099" Permanent displacement after – 0.006 to 0.020"
Type I w = 0.45 Maximum press. 60 psi	Extensive test programme (see text):
Type I w = 0.45 Gravity pressure	—

Table 1. Details from some minipile (pinpile) projects executed by companies of the Nicholson Construction Co group to June 1988.

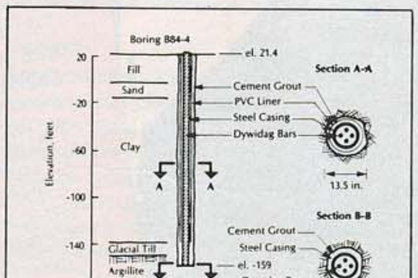
Blue Clay into the argillite bedrock. The design criteria as observed in the test and production phases were:

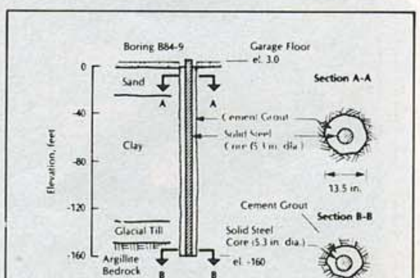
1 A design capacity of 100 tons to 175 tons.
2 Maximum pile od of 13.5".
3 Design capacity to be developed only in a rock socket formed in fresh to slightly weathered argillite, based on an allowable bond stress of 200 psi (minimum rock socket length of 10').
4 Piles could be permanently cased or not.
4A For cased piles, the following minimum design criteria were set (in accordance with the Massachusetts State Code, 1984)
*Maximum allowable stress on the steel casing of 35% fc (but not exceeding 12 600 psi). No requirements for corrosion protection.
*Maximum allowable stress on steel core (if used) of 50% fc (to maximum of 18 000 psi).
*Maximum allowable grout stress of 33% of 28 day compressive strength (to maximum of 1600 psi).
*Minimum grout thickness of 2" between core and casing, and 3" outside core in rock socket.
4B For uncased piles, the minimum criteria were
*Minimum allowable stress on the core of 50% fc (to maximum of 18 000 psi).
*No load carried by grout, above rock. Minimum crushing strength of 4000 psi in rock socket for load transfer.
*A 3" minimum grout cover over the core in soil or rock.
*Core to be centralised and extended full depth.

Construction

Over half the piles were installed from work areas in the basement with as little as 10' to 14' headroom (13' long drill mast or less). Other piles were installed from locations on top of the existing structure (42' long mast). The relatively large diameters and considerable depths led the contractor to use the high torque rotary duplex doublehead drilling system (Bruce 1984)[24], in which the rod and casing are simultaneously advanced but counterrotated.

Casing of 13.5" od in 4' 1" lengths or 20' lengths was drilled to rock head and the hole continued into rock with an 11" od rock roller bit. For the uncased piles used during production, the core comprised 5.3" diameter (175 tons) and 6.3" diameter (250 tons) undeformed solid steel bar. This was substituted in the test programme by a 9" diameter pipe of 1" wall and 7' to 10' lengths, giving a design working load of 175 tons. Minimum distances of 12" were recorded between the faces of existing walls, and pile centre.





Pile No	Type	Installed length (ft)	Max. test load (tons)	Displacement Overall	(Inches) Permanent
TP 1	Cased	182.7	500 (comp) 450 (tension)	2.338 2.370	0.494 0.687
TP 2	Uncased	163.3	500 (comp.)	1.622	0.395

Table 3. Summary of test data on test piles (TP) 1 and 2. Hynes Auditorium, Boston, MA. (From Johnson & Schoenwolf, 1987).

Neat cement grout of w=0.5 was used for grouting. Strict controls over drill flush removal were necessary in the turnpike/ railroad area and in the mechanical equipment room (Fig. 5). Thus, an outer sleeve of 16″ od was installed and sealed at the upper and lower levels to ensure the return of all drill spoil to the upper surface level, and not into the lower areas.

Given the necessity to ensure proper founding in adequate bedrock, special attention was paid in verifying the top of sound rock. These included:

* Constant updating of the rock head as determined by the site investigation data, as each pile was completed.
* Sieving of the flush return water to allow inspection of the mineral content.
* Cross hole seismics, sensing from six previously installed listening holes, to identify the special noise emitted by the rock roller on fresh rock (Fig. 6).

All three methods were used together.

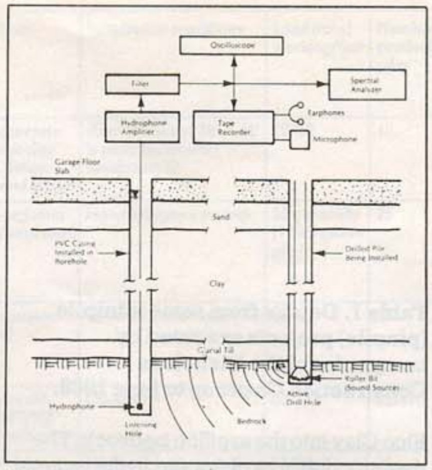


Fig. 6. Acoustical monitoring setup to aid rock head determination. Hynes Auditorium (Johnson and Schoenwolf, 1987).

Performance and testing

Both cased and uncased piles were tested, to a maximum of 500 tons (Table 3). Based on the results, the engineers concluded that such piles were feasible to design loads of 175 tons to 250 tons using the minimum recommended design criteria. The subsequent choice of uncased production piles was made by the design team based on technical submissions by the contractors, and cost comparisons.

Coney Island, New York

Background

The Coney Island Main Repair Facility of the New York Transit Authority has been in operation for 63 years and is the largest of its kind in the world. It encompasses, including the rail yards, about 100 acres, of which 12 acres are covered building space.

Constructed on the former Coney swamp, the repair shop was built on a loose fill surface with no pile support for the floors. The steel frame, columns and outside wall were supported on piled foundations. Settlement has produced major underfloor voids which have led to many floor collapses such as an 18″ drop in the main shop in 1980.

During the original construction, the swamp filling had apparently created mud waves causing uneven thicknesses of the soft organics underlying the structure.

The subsequent settlement of the ground surface due to the loading by the fill and the structure has thus been irregular in magnitude across the site.

After 'years of Band-Aids' (Munfakh and Soliman, 1987)[25] a $100M repair programme was initiated in 1984 coincident with the installation of new equipment which would alone have accelerated the settlement problem. Foundation repair had to be carried out in a fashion guaranteeing minimum disruption to shop operation, as well as constituting a proven, compatible and cost effective solution.

Remedial options under consideration included compaction grouting, chemical grouting, and concrete filled steel shell piles. However, conventional minipiles proved to be the most attractive solution from all viewpoints, and a contract was let to Nicholson Construction in early 1987 to install over 4000 piles in the fully operational facility.

Site and geology

Four distinct soil layers were identified under the slabs: Fill, peat with organic silt, grey sand and brown sand. Short and long term consolidation testing confirmed the organic layers to be the cause of the settlement. These strata experienced long term secondary consolidation and peat/organic degradation, either from oxidation or micro organisms.

Pic. 3. Special low headroom drilling equipment (8′ mast). Coney Island, NY.

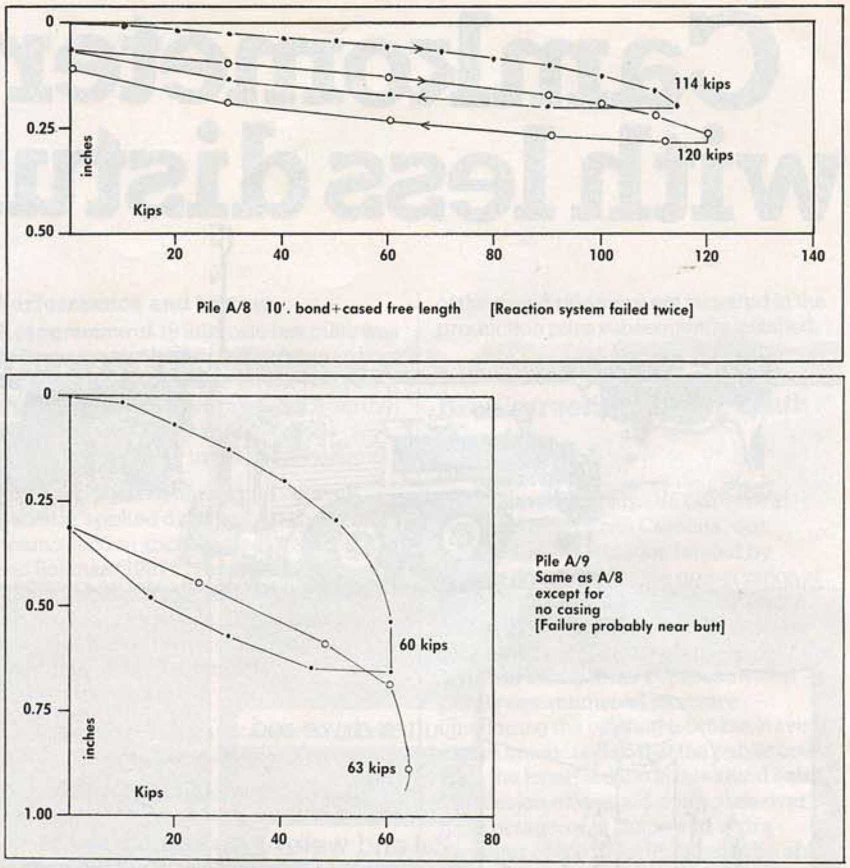


Fig. 8. Comparative test performances of two 35′ long minipiles, with and without permanent casing. Coney Island, NY.

Typically the medium dense, fine sands recognised as being adequate load bearing materials commenced 20′ to 25′ below the surface. The piezometric level was about −4′.

Access and headroom conditions were always restrictive and frequently obstructive, being as little as 8′. In addition, as the work was to be carried out in a busy, fully operational facility, in collaboration with other major structural repairs, it had to be executed in restricted 'packages' in a piecemeal fashion.

Design

Approximately 2300 number 15 ton working load piles and 1900 number 30 ton piles were required. The engineer's design allowed for the load to be taken on #6 bars, without the addition of sacrificial steel casing in the upper zones wherein resistance to buckling was analysed and judged adequate.

Standard design procedures, based on $\phi = 30°$ were used to arrive at total lengths of 35′ and 45′ for 15 ton and 30 ton piles respectively ie 10′ or 20′ into the load bearing sand.

Construction

Before installing the piles, the existing voids were filled with a lightweight foamed concrete of 30lb/ft^3 to 60lb/ft^3. It was intended that its light weight would inhibit additional settlement and corresponding downdrag forces to the piles. The fill would also protect against erosion by blocking water flow through such voids.

The access and headroom restraints over much of the site demanded the use of specially constructed drilling equipment (Pic. 3) featuring short masts and remote power units. Whenever possible, conventional crawler mounted units were

Pic. 4. Standard diesel hydraulic drill rig. Coney Island, NY.

employed (Pic. 4) with special care having to be taken in all cases with exhaust fumes and drilling spoil disposal.

The 15 ton piles were drilled and cased to 6⅝″ nominal diameter and the 30 ton piles to 7⅝ nominal diameter. Water flush was used. This casing was completely withdrawn during the pressure grouting of the sand using neat Type 1 with w = 0.50 to a maximum of 60 psi, following the placing of the reinforcing bar (6 or 9 rebar full length).

Load transfer to the existing slab structure was provided by an underreamed supporting zone formed under the slab (Fig. 7).

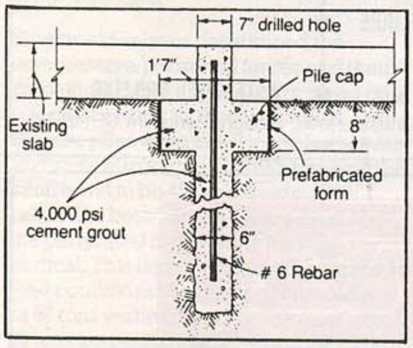


Fig. 7. Schematic arrangement of minipile and existing base slab. Coney Island, NY (Munfakh and Soliman, 1987).

Performance and testing

A programme of 10 full scale test piles was executed to verify assumptions regarding design and performance for the two pile types. PVC liners were provided from the slab to the top of the sands to ensure transfer of load only in the lower horizons.

In the first three compression tests the load was applied directly to each pile via a beam/reaction anchor system. Munfakh and Soliman (1987)[26] reported that the high concentration of stress crushed the top portion of each pile. The remaining test piles were given an enlarged cap providing better load transfer to the grout and reinforcement.

Load tests were run to twice working load in compression, and to 50 tons in tension.

The steel casing was left in place in one pile (number A/8) so that a performance comparison with the standard pile number A/9 could be obtained (Fig. 8).

Compression test data on the 6 (noncased) compression piles and the single cased pile are summarised in Table 4.

The first four piles experienced significant creep at maximum load (up to 0.35″ in 4 hours) whereas those tested through the cap had less (0.032″ to 0.064″ in 4 hours at 30 tons). The cased pile had less than half this amount of creep in 5 hours at 30 tons.

Such performances were acceptable to the structural designers and the benefits of the cased pile were not required in the production piles subsequently installed.

Pile #	Description	Ratio of grout volume to hole volume	Stiffness in *lineal part (tons/inch)	Max. load & total accum. deflection (tons)	(inches)	Notes
A/3 A/4	Loaded annulus only	1.2 3.7	80 85	20 (F) 31 (F)	1.25″ 0.65″	Failure premature and most probably due to crushing of pile head.
A/5 A/9	Loaded full section	2.5 2.9	95 72	29 (F) 31 (F)	0.75″ 0.85″	Failure possibly due to soil/grout failure although distress at head also noted.
A/7	Includes original conc. slap in cap	2.9	303	70 (F)	0.90″	Soil-grout failure likely.
A/10	Excludes conc. slab in cap	3.4	178	56	0.42″	Test suspended upon failure of pile cap.
A/8	With sacrificial casing for 25′	7.7	385	60	0.30″	Test suspended when reaction pile pulled.

TABLE 4

* A measure of pile stiffness obtained by dividing the maximum load over which displacement is relatively linear, by the displacement at that load.

Table 4. Comparative performance of 15 ton working load piles, Coney Island, NY. All piles were 6⅞″ in diameter, 35′ long including 10′ bond, and had a full length #6 rebar.

Brookgreen Gardens, South Carolina

Background

The 300 acre Brookgreen Gardens at Murrell's Inlet, South Carolina, was founded as an institution funded by private donations for the preservation of the flora and fauna of that southeastern part of the United States. A scheme was conceived to construct a large and almost invisible aviary in the cypress swamp – a purely environmental structure enveloping the existing trees but leaving nature untouched so that the public could view the local bird life in its natural habitat. The design envisaged a structure over 90′ high, octagonal in shape and with a diameter of 200′ (Fig. 9). It had to be able to withstand hurricane winds, ice storms and a marine atmosphere. It had to be constructed in a swamp without damage to the existing trees or character of the swamp, with a tight budget and in a very short period.

The walls of the aviary were made of a special polyester safety netting, suspended off nine slender aluminium poles. Each pole was supported by two cable stays, and prestressed ground anchors were chosen to provide ground fixity for these. Equally, however, the poles exerted compressive stresses on the ground, while dynamic analysis indicated that the pole foundations would also be subjected to significant horizontal forces under certain circumstances. Conventional piling and spread footings were unacceptable because of the nature of the site and the potential disturbance to the existing flora. Minipiles were the logical choice, particularly so since the equipment (and techniques) needed for the soil anchors, could be used economically for the piles also.

Site and ground conditions

The nature of the site is illustrated in Pic. 5. Timber planking had to be set as temporary access into the swamp from the main road.

Site investigation holes showed grey/brown loose fine sands, under the cypress roots, overlying dense medium and coarse light brown sands with shells. Water level coincided with the ground level.

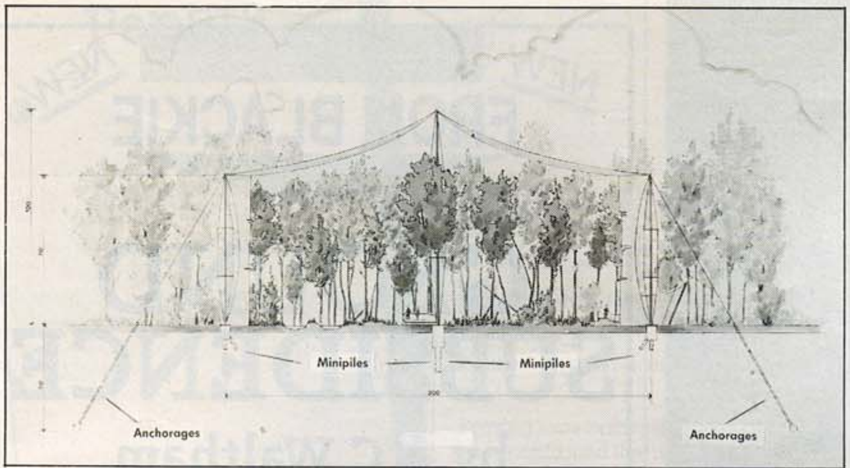


Fig. 9. Artist's impression of the aviary in the cypress swamp, showing minipiles underpinning footings, and ground anchorages resisting tensile cable loads. Brookgreen Gardens, South Carolina.

Design

Cable tensile forces were reacted by bar tendon anchorages 55′ deep at 45° to the horizontal. The maximum working load of each of these eight anchorages was 42 tons. Eight 9 ton wall tie down anchors were constructed in similar fashion at 55° to the horizontal.

Structural analyses determined the compressive/horizontal forces and their resolution. Each mast was supported on a pile cap bearing on three minipiles. Two of these piles were at 45° to the horizontal and were also aligned in plan towards the centre and to be 45° either side of the radial line between the centre mast and the peripheral mast. The third was vertical. This layout satisfied the maximum load conditions that could occur, defined as 55 tons vertical load at each mast pile, 21 tons horizontal load towards the centre mast, and 11 tons horizontal load towards the adjacent peripheral mast.

The external and internal load carrying capacities of the pinpiles were determined in standard fashion.

Construction

The 55 ton working minipiles were constructed in a similar fashion to the tie down anchors, except that a 33′ long pressure grouted zone was used for the angled piles and 10′ for the vertical. In each case a 20′ length of 5″ od steel casing of minimum wall thickness 0.312″ was left in place for the length above the pressure grouted zone. Pressures of up to 120 psi were used with neat Type 1 cement grout of $w = 0.50$. Each pile was further reinforced full length with one number 9 rebar with centralisers. Four number 4 hooked rebars were also set in the fresh grout at the top of each pile. The three minipiles were then connected into a pile cap which consisted of a piece of 42″ diameter steel tubing set around the projecting steel pile casings and reinforcing bars. Further layers of W6 x W6 reinforcing steel mesh were also placed and the cap filled with 4000 psi concrete. Holding down bolts for the articulated mast bases were held by templates and also concreted in.

The 15 ton centre mast pile consisted of a variation of the standard minipile in that the total length of the single pile required was 20′ of which the top 10′ was a piece of 18″ diameter pile casing. This arrangement was chosen as the maximum vertical load calculated was much less than the peripheral piles and no horizontal forces were anticipated.

A wide track diesel hydraulic drilling rig was used throughout with a rotary duplex drilling system with water flush. The return flush and cuttings were ponded carefully and led away. Special precautions were likewise taken with the mixer pump unit and ancillary equipment.

At every stage of the operations, the curator of the gardens was consulted as to the impact of each construction step on the enclosed area.

Performance

All the objectives of the project were achieved. The scheme – an engineering joint venture between the architect (Clarke and Rapuano Inc.) and the contractor (Nicholson Anchorage Co.) received first prize in that year's New York Association of Consulting Engineers Engineering Excellence Competition. Judgment was based on project significance, complexity, uniqueness, clients' needs, budget, originality, value to the profession, and timeliness.

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To be concluded in Ground Engineering January 1989.

Aspects of minipiling practice in the United States

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Continued from *GE* November 1988. This is the conclusion of Bruce's description of a decade of minipiling projects in the US.

Warwick, NY

Background

The A Alfred Cohen Physical Education Building at the Mid Orange Correctional Facility in Warwick, New York, was completed in 1972. Eight years later differential settlement of the structure had occurred to a degree and at a continuing rate which demanded major remedial action to arrest the condition. At this point the damage was most severe along portions of the north and east walls of the gymnasium building, although the swimming pool structure was also under threat.

The major element of the repair programme was the use of rather exceptional minipiles to underpin about 135′ of load bearing walls and columns in the north and east of the structure. The minipiles were to be installed vertically through the existing footing, only 1′ thick and founded about 8′ below the existing ground surface. To ensure proper load transfer between the new piles and the structure, the piles were to extend upwards into a new reinforced concrete pile cap, which had to be horizontally post tensioned with Dywidag bars to provide

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additional clamping (Fig. 10). A total of 62 piles of working load 27.5 tons was designed, based on the stipulation that they support the entire dead and live loads. This support scheme also had to guarantee not more than ¼″ additional settlement within two years. Given the relatively long pile lengths necessary to reach a suitable bearing stratum, this performance could only be provided with certainty by preloading the piles, and releasing the load after the casting of the new beams.

The overall repair programme also included concrete patching and resin injection of existing beams, as well as other miscellaneous remedial activities associated with the piling. The contractor, Nicholson Construction Co., was also responsible for the design of the whole underpinning system.

Site and ground conditions

Piles were installed from inside with 20′ headroom, and outside the structure.

The soil proved to be loose/medium brown silty sands with gravel and clay overlying medium dense compact gravelly sand, occasionally silty, at depths below about 40′. The water table was variable but was typically 15′ to 25′ below the surface.

Design

A 25′ long bond zone was determined using the standard empirical approach from ground anchors, and assuming $\varnothing = 35°$, grouting pressures of around 100 psi and an assumed effective bond diameter of 8″.

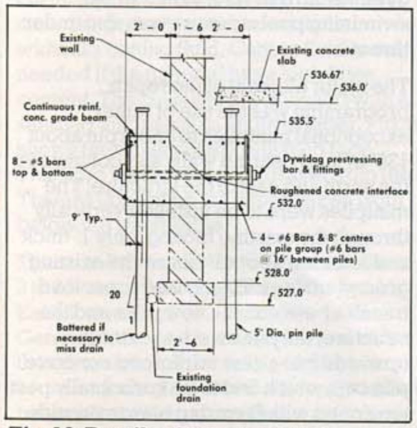

Fig. 10. Details of attachment of minipiles to existing structure. Mid Orange Correctional Facility, Warwick, NY.

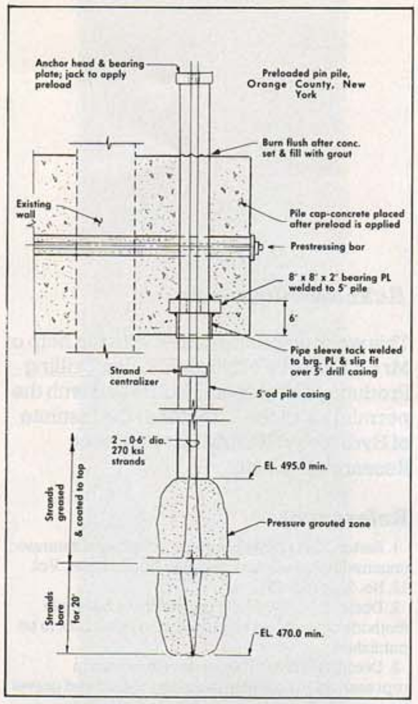

Fig. 11. Detail of preloaded minipile, Mid Orange Correctional Facility.

Stress calculations showed that a 80 ksi, 5″ diameter pipe of 0.362″ wall thickness would resist safely the maximum test load anticipated.

To allow preloading, two 0.6″ diameter prestressing strands were installed beyond the tip of each pile. Each strand was bonded over the lower 20′ and sheathed up through the pile to the head (Fig. 11).

The design of the new capping beam was checked for lateral bending, longitudinal bending and adhesion to the existing wall. This aspect revealed the need for horizontal clamping, the roughening of the existing walls and the use of a concrete bonding agent to satisfy the design criteria.

Construction

The 5″ casing was drilled from surface elevation 537′ to full depth. The two strands were placed and the casing tremied full of neat cement grout of $w = 0.45$. Pressure grouting was then conducted to a maximum of 80 psi to 120 psi back up to elevation 490′. Grout takes ranged from 15 to 42 bags per pile, but were typically close to the 25 bags average. The permanent casing was then pushed back down to elevation 495′. Once the grout had reached an unconfined compressive strength of 3000 psi, preloading was carried out to 50% of the design load.

The concrete beam was then cast and post tensioned against the existing structure. Thereafter, the preload was released and so the structural load was assumed by the minipiles.

Testing and performance

Two load tests were conducted to 55 tons using adjacent piles as reactions. The total settlements at 55 tons were 0.188″ and 0.249″, while creep movements over four hours at that load were 0.002″ and 0.005″ respectively. Permanent displacements were of the same order.

Column settlements were monitored regularly throughout and after the work. Readings at 24 locations showed a maximum of 0.01″ of additional settlement during and just after the whole reconstruction programme. Since then no further settlements have been recorded.

Warren County, NJ

Background

The I-78 dual highway crosses the Delaware river between Pennsylvania and New Jersey (Warren Co) on seven span, multigirder bridges (Pic. 5). Generally foundations on the Pennsylvania side incorporated driven H piles whereas the river piers and the New Jersey piers were foreseen as founded on solid rock. This proved to be valid except for pier E-6 on the eastbound structure.

Excavation for the footing to the planned elevation had revealed that rock was nonexistent. Further excavation to an elevation 15′ to 20′ below revealed only random rock thicknesses of several feet and a highly irregular bedrock surface. The excavation was filled with lean mix concrete and the foundation design reconsidered.

Pic. 5. Installation of minipiles to underpin bridge pier 6E over the Delaware river. Warren County, NJ.

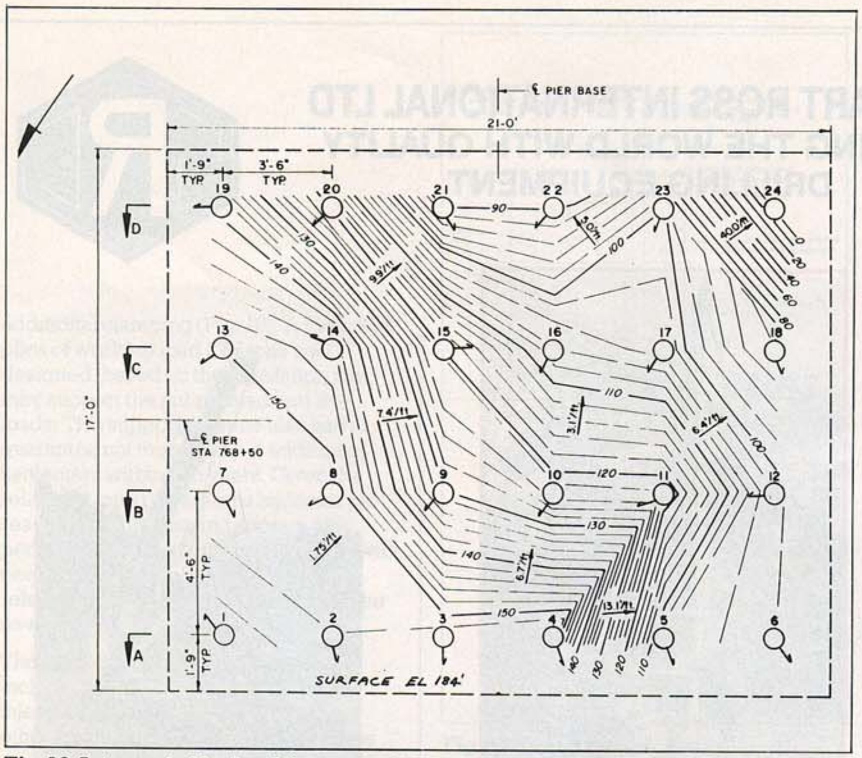

Fig. 12. Interpreted bedrock isopachs. Warren County, NJ. Arrows show direction of drill hole deviation (See *Table 5*). Data courtesy of Modjeski and Masters.

Various options reviewed included:
*Enlarged spread footings
*H piles in predrilled holes
*Elimination of the pier
*Relocation of the pier
*Deep bored piling.

Only the last option proved feasible and two alternates were considered:

(i) Six large diameter (36″) caissons each of working load 360 tons
(ii) 24 minipiles each of nominal working load 100 tons (allowing a 11% redundancy, reflecting the highly variable rock conditions).

Bids were solicited for each option, but due to the extremely onerous geological and programming restraints, only one contractor for each responded. The bid for the 36″ diameter caissons was essentially cost plus with a guesstimated figure of about $1M. Nicholson's fixed price offer for the minipiles was less than half that figure. The owner therefore decided on the latter option on grounds of cost, programme time and the ability to demonstrate the effectiveness of the system by a test pile installed in advance.

A further technical advantage was the action of minipiles in transferring load by skin friction as opposed to end bearing: the possibility of pile failure by punching through into any soft underbed immediately under founding level was therefore eliminated.

Site and ground conditions

The bedrock was a Cambro-Ordovician dolomitic limestone referred to locally as the Allentown Limestone. It proved to be moderately/highly fissured, cherty, and very susceptible to karstic weathering. Major clay filled beds were intersected even over 100′ below the surface eg 50′ of soft brown silty clay below 106′ at pile 24. Dipping 55° to the southeast, the rock mass proved highly variable laterally and vertically. The solid bedrock surface, as revealed in site investigation holes, and by the subsequent pile drilling is shown in Fig. 12.

Design

The owner's design regulations permitted:

*Maximum average rock/grout bond at working load (100 tons) of 50 psi.
*Maximum allowable reinforcement steel stress (fa) at working load equivalent to 45% fc.

These factors led to the selection of:

*A load transfer zone, 8½″ diameter and 15′ long in competent rock.
*Use of a 55 ksi low alloy steel pipe of od 7″ and wall thickness 0.408″ as pile reinforcement.

Recognising that the rock was likely to be very variable, provision was made to allow the 15′ bond zone to not necessarily be continuous. In most piles this was subject to the following restrictions:

*The lower part of the zone to contain at least 10′ of continuous sound rock
*Soft interbeds to be less than 3′ thick
*A zone of acceptable load bearing rock to be at least 5′ thick
*Regrouting and redrilling of interbeds within the overall bond zone to be undertaken

Piles 1, 6, 17, 18, 19, 23 and 24 were required to have a continuous 15′ bond zone.

Construction

The trackrig of Pic. 5 was used for all drilling operations. The sequence of installation was as follows:

*Install 10.75″ od casing through the backfill and socket into the concrete of the cap.
*Drill with 10″ down the hole hammer through the concrete footing.
*Install 9.625″ casing through the less competent upper horizons (normally 30′ to 45′). Survey linearity and grout in place.
*Drill 8.5″ hole by hammer or rotary to ensure minimum of 15′ bond zone as described above.
*Flush hole and install 7″ od reinforcing pipe. Survey for verticality, not more than 2% deviation allowable.
*Tremie grout hole pile and pressure to 50 psi.

Verification of each pile alignment was made through the use of an R single shot

Pile	Length (FT)	Actual drift (inches)	Ratio actual to allowable deviation (based on 2% deviation)	Direction of Drift (see figure 12)
1	44.0	4.6	44%	S 50°E
2	47.0	3.45	31%	N 45°W
3	46.0	6.13	57%	N 30°W
4	45.0	2.36	22%	N 85°W
5	93.0	1.95	9%	N 77°W
6	97.0	8.10	35%	S 85°W
7	49.0	6.11	53%	N 57°W
8	49.0	4.04	35%	N 05°E
9	67.0	8.21	70%	N 18°W
10	77.0	9.68	52%	N 13°W
11	77.0	5.64	30%	N 14°E
12	97.0	10.16	44%	N 32°E
13	49.0	5.13	43%	N 85°W
14	52.0	9.80	78%	N 75°E
15	80.0	5.03	26%	S 45°W
16	93.0	13.60	61%	N 20°W
17	96.0	2.00	9%	S 4°W
18	107.0	4.45	17%	N 70°W
19	60.0	5.91	41%	N 45°E
20	80.0	11.73	61%	N 10°W
21	108.0	11.20	44%	S 61°W
22	109.0	9.13	34%	S 12°E
23	98.0	10.26	44%	N 12°W
24	200.0	14.24	35%	–(22′ above base)

Average = 40%
ie average deviation of less than 1%

Table 5: Borehole deviation data on minipile holes, Warren County, NY.
(Data courtesy of Modjeski and Masters)

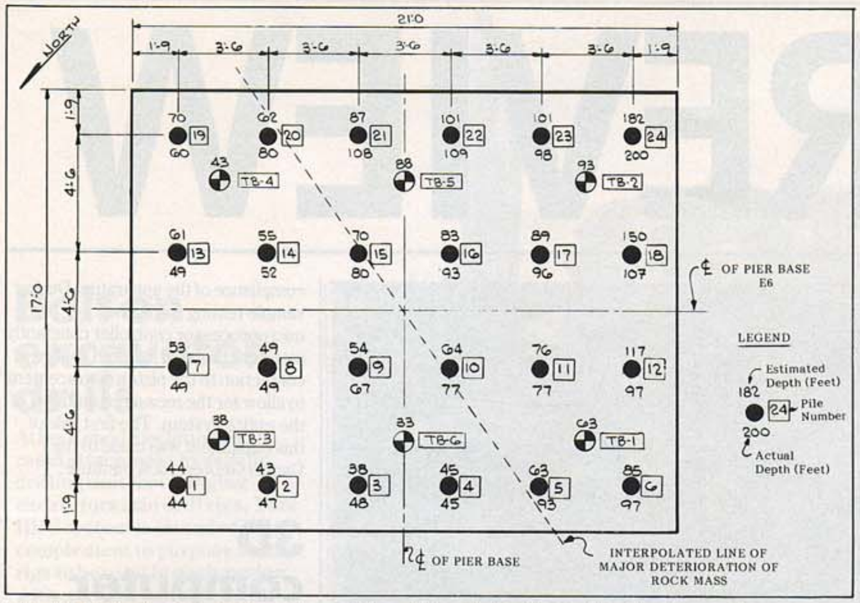

Fig. 13. Actual minipile lengths, with anticipated caisson depths shown for comparison. Warren County, NJ.

direction survey instrument, manufactured by Eastman-Whipstock. Each pile was surveyed at top, bottom and mid depth. The results are shown in Table 5 and these indicate every pile fell within the criteria, with most being within 1% deviation.

Grout was mixed in a collodial mixer and injected by Moyno pump. A neat Type III mix of w = 0.5 was used providing three day crushing strengths of over 3500 psi.

Throughout construction, the very adverse conditions posed major drilling problems. These were resolved, at length, by repeated pregrouting and redrilling. Great care was taken to provide bond zones in accordance with the design provisions. Fig. 13 summarises the actual total drilled lengths.

Regarding the anticipated caisson tip elevations, also shown in Fig. 13, these would have been in all cases shorter than subsequently proved necessary to found safely the minipiles. Poor or voided rock was consistently found below these anticipated elevations, further supporting the decision to use minipiles.

Overall the total drilled length of 1920 linear ft. corresponded with the total foreseen quantity of 1710 linear ft. Variations from 43′ less to 30′ more with respect to foreseen were recorded on individual piles, highlighting the variability of the rock. Overall, a volume of grout equivalent to four times the nominal hole volume drilled was injected. Much of this was consumed in the zone above rockhead during pregrouting operations. The level of maximum takes corresponded with groundwater level.

Testing and performance

A separate test pile, 30′ long with only 5.33′ of bond was load tested in accordance with ASTM D1143 quick load test method to 205t, using rock anchors as reaction. This particular socket length was selected as at test load, the average grout to rock and grout to steel bonds would be 304 psi and 250 psi respectively – both considered to be at or near ultimate values. An outer sleeve of PVC pipe extending to the top of the rock socket ensured load transfer only in the socket. A 6″ thick wooden plug was attached to the bottom of the steel pipe to ensure no load could be transferred in end bearing.

The data is presented in Fig. 14. In summary the total settlements recorded at each successive cycle to 205t were 0.367″ and 0.373″ respectively. Creep of 0.011″ was recorded over 60 minutes hold at these loads. The permanent set after this operation was 0.07″.

The next day testing was continued to higher levels, but at 224t the material of the upper casing began to fail. Until that point, the pile was performing exactly as it had during the previous testing sequence. Total displacement was 0.371″ at 215t, but 0.452″ at 224t.

During installation of the reinforcing pipe in the last and deepest pile (No 24), a thread parted and a 130′ length of pipe fell into the 200′ deep hole. Borehole TV revealed the casing to be further ruptured 30′ above the bottom of the hole, due to its impact with the bottom. After various attempts at recovery, it was decided to grout the pile, having previously suspended a 20′ long, 4½″ diameter 150 ksi steel pin, with centralisers, from 62′ to 82′ below the top. The intention of this pin was to ensure effective load transfer across the upper discontinuity. A very rigorous extended load test was then executed to 170t. The performance of the pile proved excellent. Total displacement was 0.187″ at 170t, 0.010″ creep in 24 hours, permanent set 0.009″. It was judged capable to safely perform its function in service.

The bridge is now complete and the performance of pier 6E has proved exceptional.

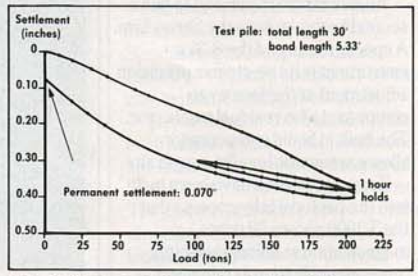

Fig. 14. Test data on test pile. Warren County, NJ.

Final remarks

The case histories described in this paper highlight clearly the vigorous and expanding nature of the minipile market in parts of the United States.

In addition, it would appear that, in design and construction, American practitioners are proving at least as innovative and resourceful as their counterparts in other parts of the world with significantly longer histories of minipile applications.

As the current trend for urban and industrial remediation and redevelopment continues to gather momentum, it is easy to predict with confidence a healthy growth for the technique of minipiling in the United States.

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