Pinpiling in the United States -
Practice and Potential

by

Dr. Donald A. Bruce*

SUMMARY

Small diameter cast in place bored piles (pin piles) have been used around the world for almost 40 years. The main application has been to underpin existing structures to arrest settlement. The paper describes their construction, design, and performance, and outlines some development trends. Two major case histories from the Eastern States are reviewed. It is possible to conclude that the recent expansion in the pin pile market in this country will continue apace in the light of current construction trends, and an increasing appreciation of the great potential of the technique.

1. INTRODUCTION

It is now almost 40 years since the technique of minipiling - known as pin piling in the United States - was first applied in Italy. Following the expiry of the original patents in the early 70's, the popularity of the system spread rapidly throughout Western Europe and certain Far Eastern countries (Bruce, 1988a). This growth reflected the trend of the construction market in these areas: an emphasis on infrastructure development, redevelopment and upgrading in areas of high population density.

Pin piles were, therefore, used to prevent or arrest structural settlements generated by the construction of adjacent excavations or tunnels, by changes in the ground water level, by changes in foundation loadings (by additions), or by the imposition of machine vibrations to structures and foundations. In most applications, the piles had to be installed with minimal environmental damage, and had to provide exceptional load/settlement characteristics.

In the United States, the particular ramifications of undertaking complex ground engineering works in existing urban and industrial environments have impacted rather later (Bruce 1988b). For example, 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 tunneling, 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.

*Technical Director, NiCon Corp., P. O. Box 1357, Pittsburgh, PA 15243

Likewise the major growth in pin piling in the U. S. has occurred since the late 70's, to the extent that today there is probably a greater intensity of pin piling in the Boston/New York area than anywhere else in the world. As an indication of the range of projects undertaken by the author's company alone, Table 1 lists jobs conducted over the last few years. These examples exclude the frequent cases where pin piles have been used as reactions for ground anchorage tests, or where they have been installed as in situ reinforcement for slope stability (Boley and Crayne, 1985) or where they have acted as simple "pins" to stabilize the toes of sheet pile walls.

2. CONSTRUCTION

The most common basic method of installing pin piles is shown in Figure 1. Variants using compressed air to pressurize the grout, or a vibrated mandrel (displacement pile), are described by ASCE (1987) but are largely obsolescent. In addition, the "expanded base" pile (Lizzi, 1982) and the Menard inflatable cylinder pile (Mascardi, 1982) are likewise seldom seen nowadays.

Although certain authors (e.g. Weltman, 1981) try to differentiate and classify minipiles or pin piles in terms of diameter, it is perhaps more appropriate to regard all such piles that can be installed with conventional drilling and grouting equipment as pin piles. This definition normally limits diameter to about 12" within the normal depths involved (less than 100').

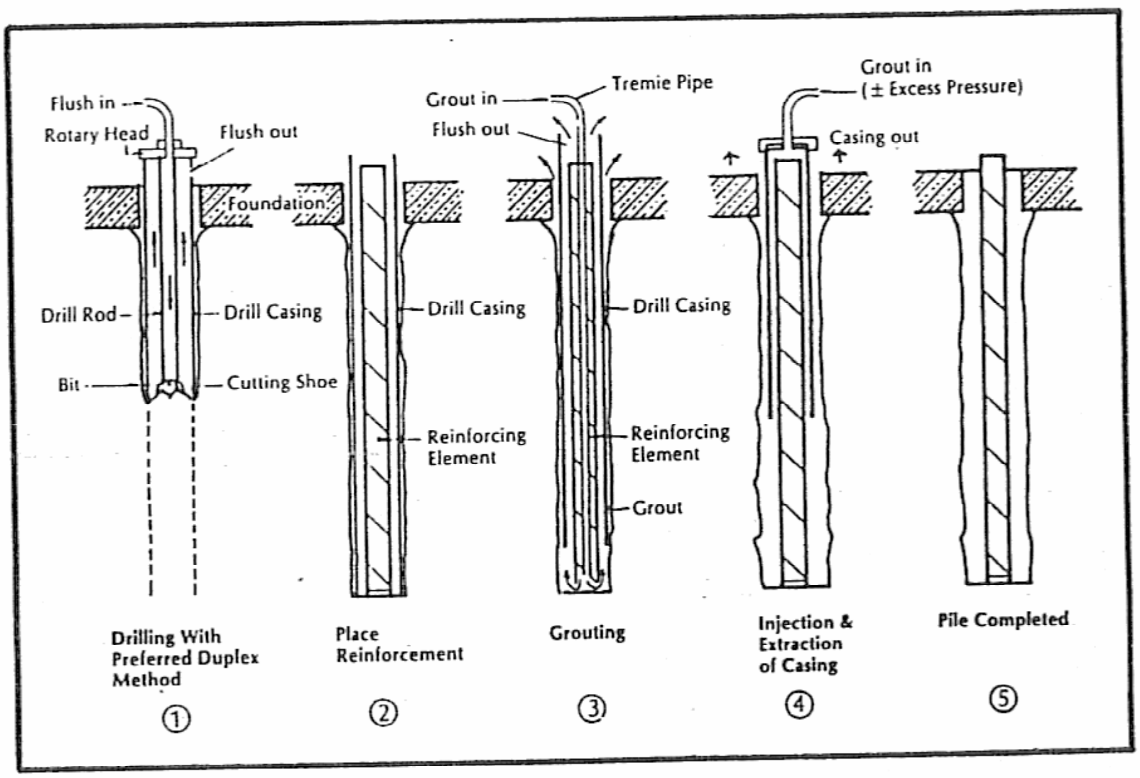

Figure 1. Stages in the construction of a standard pin pile (After Koreck, 1978).

LOCATION	LOCATION/APPLICATION FOR FOUNDATIONS BEING UNDERPINNED	FOUNDING STRATA	INSTALLATION HEADROOM	COMPRESSIVE LOAD (TONS) WORKING/TEST	NR. OF PILES	TYPICAL LENGTH (FT)
Apollo, PA	New tank in existing wastewater treatment plant.	Dense sand & silt/gravel	18'	10/20	45	30
Brookgreen Gardens, SC	Supporting masts of suspended net forming "natural" aviary in swamp, with minimal damage to environment.	Med.-dense sand	Swamp	55	25	35
Neville Island, PA	Existing dust collector structure on rapidly compacting soil.	Sand & gravel	10-16'	30/60	32	29
Providence, R. I.	Test to assess viability of underpinning existing granite block seawall.	Sandstone	Open air	55/110	TEST	65
Trafford, PA	New printing press in existing building.	Weathered shale	14'	10/20	20	36
Warwick, NY	Existing gymnasium building (use of preloaded piles).	Glacial till	20'	27.5/55	62	65
Monessen, PA	Existing operating coke battery, Emission Control Facility.	Clayey sand & gravel	19-25'	50/100	102	65
Mobile, AL	Two existing sodium hydroxide storage tanks under which wood piles had failed.	Dense sand & gravel	8-15'	34 and 54	171+7	55

Table 1 Continued

LOCATION	LOCATION/APPLICATION FOR FOUNDATIONS BEING UNDERPINNED	FOUNDING STRATA	INSTALLATION HEADROOM	COMPRESSIVE LOAD (TONS) WORKING/TEST	NR. OF PILES	TYPICAL LENGTH (FT)
Burgettstown, PA	Existing gantry runway.	Sandstone, limestone	24'	10	20	32
Dunbar, PA	Addition to water treatment plant.	Sandstone	Open air	45	7	26
Pittsburgh, PA	Existing structure adjacent to deep excavation.	Dense sand & gravel	Open air	50	21	30
Pittsburgh, PA	Existing parking garage.	Sandstone	8-10'	55	46	43
Aliquippa, PA	New Emission Control Building at existing coke battery.	Dense sand & gravel	25'	50/100	39	75
Jeannette, PA	New machine in existing building.	Bedrock	20'	Various	27	35
Apollo, PA	New nuclear power structure in existing building.	Sands & gravel	20'	10	24	23
Marion, IN	Existing body stamping plant.	Bedrock	18'	60	24	70
ALCOA, TN	New building in existing rolling mill.	Limestone	Open air	70/140	93	40

Table 1 Continued

LOCATION	LOCATION/APPLICATION FOR FOUNDATIONS BEING UNDERPINNED	FOUNDING STRATA	INSTALLATION HEADROOM	COMPRESSIVE LOAD (TONS) WORKING/TEST	NR. OF PILES	TYPICAL LENGTH (FT)
Washington DC	Existing structure at Castle Building, Smithsonian Institute.	Dense sands & gravel	Very restrictive	50/100	21	75
Pittsburgh, PA	Restoration of existing Timber Court Building	Sandstone	10'	50	15	70
Warren Co., NJ	New bridge pier.	Limestone	Open air	100/224	24	78
Kingsport, TN	New storage tank in existing building.	Limestone	11	40/80	115	35
Boylston St. Boston, MA	Existing building being redeveloped.	Sand	8	40/92	262	27
Ann Street Pittsburgh, PA	To support new soldier beams for new retaining wall.	Sandstone	Open air	45/68	86	12
Coney Island, NY	Rehabilitation of existing repair shop.	Dense sand	8'	15/30 & 30/60	2300 + 1900	35 45
Cleveland, OH	New addition to existing Control Building	Shale	Open air	60	45	140

TABLE 1. Summary details of some pin pile contracts executed by compaines of the NiCon Corporation.

The details of construction vary, but certain factors should remain common:

- Drilling: the method must cause minimal disturbance or upheaval to the structure or soil.

- Grouting: relatively high strength neat cement or sand-cement grouts are used at injection pressures rarely above 100 psi.

- Reinforcement: may be reinforcing cages (compressive loads only), high strength bars (compression or tension) or pipes (to resist bending stresses).

- Connection to structure: adequate bond or connection must be provided in order to properly transfer loads. In the case where the load must be transferred solely by bond within the existing structure, the (normally smooth) structural interface can be roughened to give additional mechanical interlock in order to ensure adequate load transfer. Such a system, termed Ankerbonder, has recently been used on a major underpinning project for cooling towers in England (Anon, 1987).Employing vibrating air driven pistons with tungsten carbide tips, the head is lowered into the (diamond drilled) hole and rotated slowly. The typical groove configuration is shown in Figure 2. The resultant roughened interface gives ultimate structure/pile bond values up to ten times higher than conventional systems.

- Corrosion protection: when piles are required to act in tension, or when they are installed in particularly aggressive conditions, then particular attention must be paid to the corrosion protection of the steel element. Similar to ground anchorages, protection in the form of an outer corrugated sheath can be used, while it is prudent to centralize the steel in the hole to ensure that a minimum grout cover of about 1" is provided to the steel.

3. DESIGN AND PERFORMANCE

Pin piles are designed to operate by side friction as opposed to end bearing, due to their geometry: they are very slender elements in which the lateral area is typically hundreds of times larger than the base. Their geometry partly explains their excellent load/settlement characteristics: the relative displacements needed to mobilize frictional resistance are much smaller than those needed to develop end bearing.

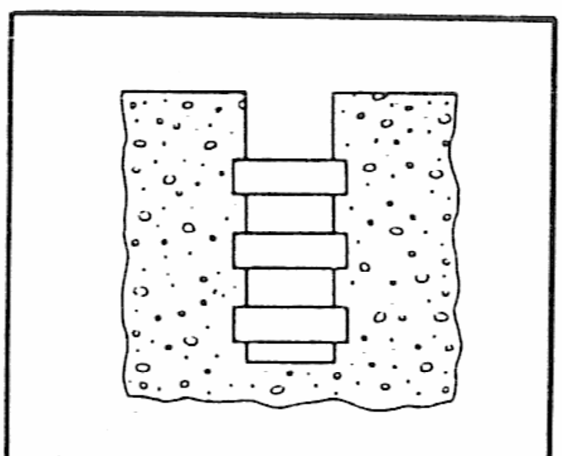

Figure 2. Typical triple-groove bonding profile produced by the Ankerbonder in a diamond cored hole. At the West Burton project, in England, two sets of triple grooves were provided per hole. (Anon, 1987)

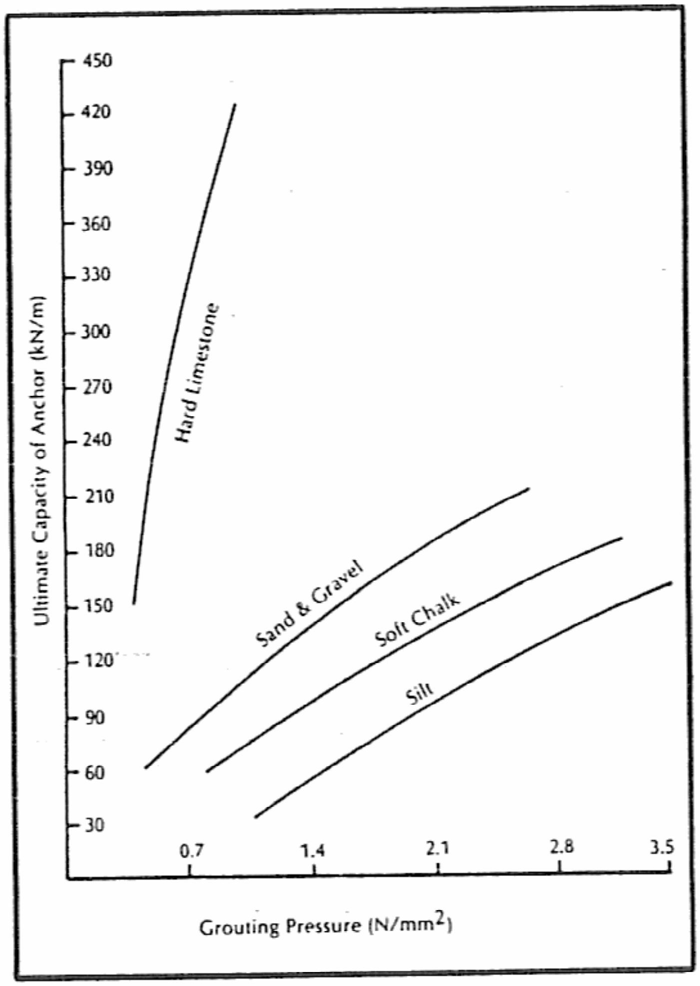

Figure 3. Influence of grouting pressure on ultimate load holding capacity. (Littlejohn and Bruce, 1977)

In addition, the method of their construction, and in particular the use of high-strength grouts injected at significant pressures, acts to promote excellent bond characteristics with the soil. Analogies can be drawn with soil anchor practice (see Figure 3), albeit for interfaces in the opposite sense of shear. As a general guide to the design of the transfer length, the recommendations of PTI (1986), pages 27-30, may be followed.

With these major points in mind, the basic design philosophy differs little from that for any other type of pile: the system must be capable of sustaining the anticipated loading requirements within acceptable settlement limits, and in such a fashion that the elements of that system are operating at safe stress levels. In detail, attention must be paid analytically to settlement, bursting, buckling, cracking and interface considerations whereas, from a practical view point, corrosion resistance, and compatibility with the existing ground and structure (during construction) must be regarded. The system must also be economically viable.

Reference must, therefore, always be made to local construction regulations for guidance, although the special aspects of pin piles may not be adequately or specifically addressed. In that event, sensible interpretation is necessary.

Generally, it is found that whereas the design of a conventional system is normally controlled by the external (i.e., ground related) carrying capacity, their small cross sectional area dictates that pin pile design is most often limited by the internal carrying capacity. Emphasis is, therefore, placed on the steel and grout parameters as well as the grout/steel bond.

Regarding the internal stability of pin piles, mathematical models can be called upon to investigate the stability of pin piles with respect to buckling and bursting resistances. Regarding the former, early work by Bjerrum (1957) is supported by the detailed analyses of Mascardi (1970, 1982) and Gouvenot (1975). All authors conclude that only in soils of the very poorest mechanical properties, such as loose silts, peat and non-consolidated clays, where E is less than 70 psi, is there even a possibility of failure through insufficient lateral restraint.

Similarly, bursting can typically be discounted, but where the possibility does exist, additional lateral restraint can be provided by increasing the thickness of the grout annulus, modifying the grouting design and methods, increasing the spiral reinforcement or by maintaining a permanent casing through dubious horizons.

A final point may be made in relation to the "group effect" - a vital consideration in pin pile design. The contrast with conventional piling is fundamental. For example, the British Code of Practice 2004 (1972) states that for "friction piles, the spacing centre-to-centre should be not less than the perimeter of the pile; with piles deriving their resistance mainly from end bearing, the spacing centre-to-centre should be not less than twice the least width of the pile." This spacing is to avoid the "negative" group effect.

On the other hand, Lizzi (1982), and others, including ASCE (1987), refer to the "knot effect" in pin piles whereby a "positive" group effect is achieved in the loading of the soil-pile system. For example, Plumelle's full-scale testing (1984) yielded the results shown in Figure 4 that confirmed Lizzi's earlier model tests (see Figure 5). The latter noted that the increase was proportionally greater in the sand than the cohesive pozzolanic material that allowed interaction in even the Group A arrangement. This "knot effect" cannot be relied upon when preloaded piles are used.

4. DEVELOPMENT TRENDS

In the last few years, as the market has increased and the number of geotechnical specialist contractors qualified to do the work has grown, several innovations have been made. These have been directed towards providing piles of superior performance more cheaply and in more challenging structural or environmental settings. Three topics merit special attention:

- Post-grouting of bond zone
- Reinforcement of free zone
- Preloading

By injecting cement grouts into the bond zone after the first stage grout has set, a significantly improved load bearing performance can be provided. The injection of cement grouts can be accomplished via a separate grouting tube (i.e., a sleeved pipe) or by using the steel reinforcement itself as the grout pipe. This method is used in the TUBFIX and ROPRESS type piles, wherein the packer is introduced into the steel core pipe and grout is ejected through the

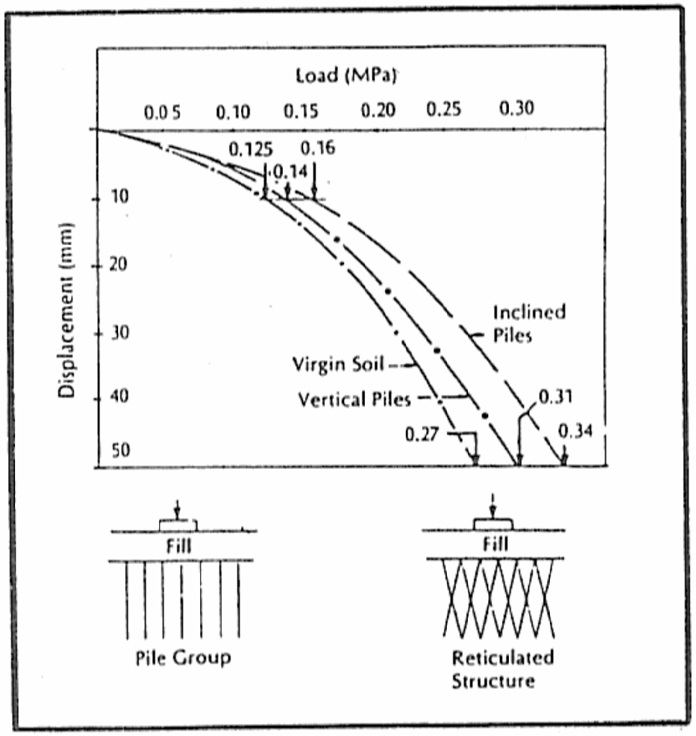

Figure 4. Field test data for different pin pile arrangements. (Plumelle, 1984)

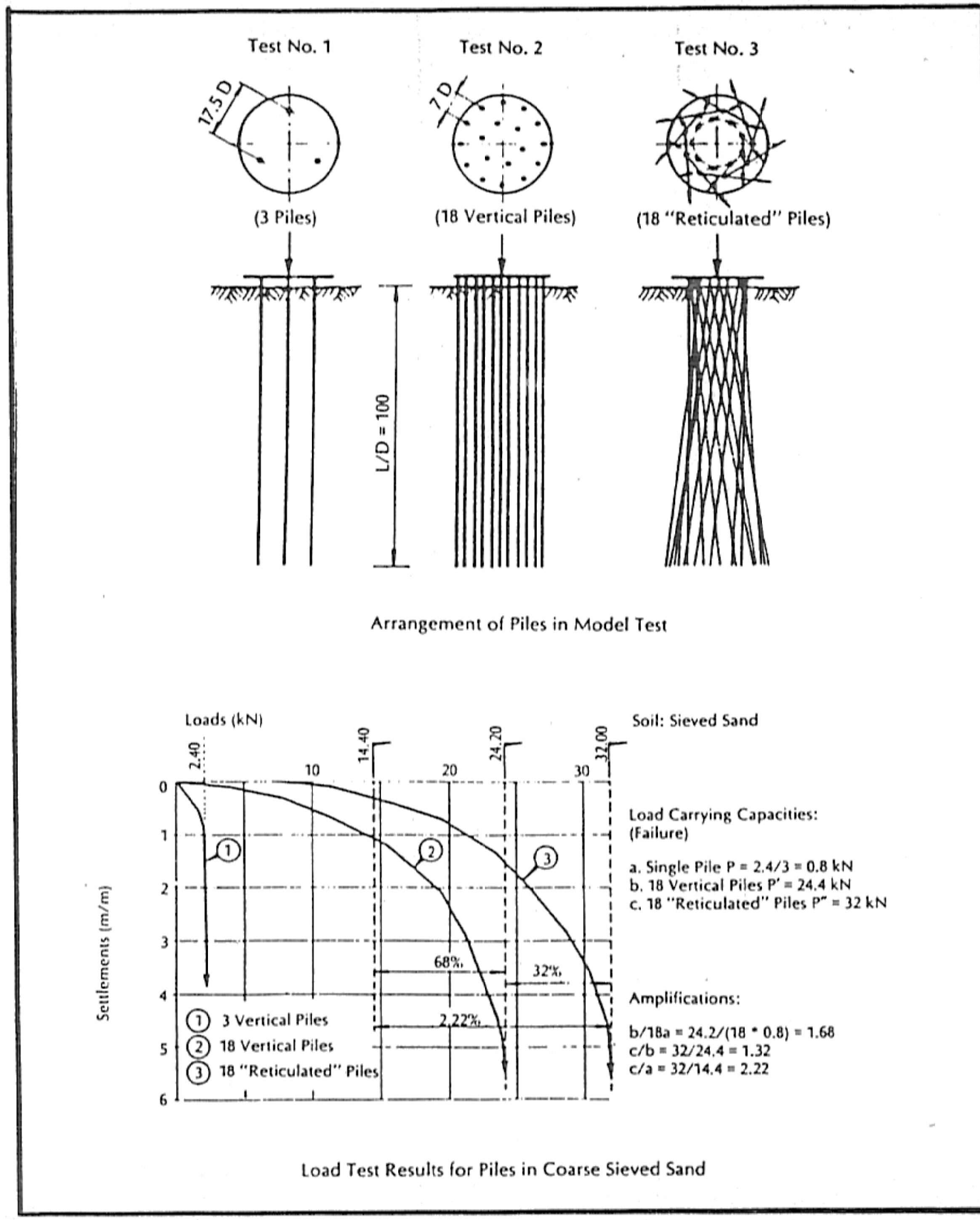

Figure 5. Model test data for different pin pile arrangements in coarse sieved sand. (Lizzi 1978)

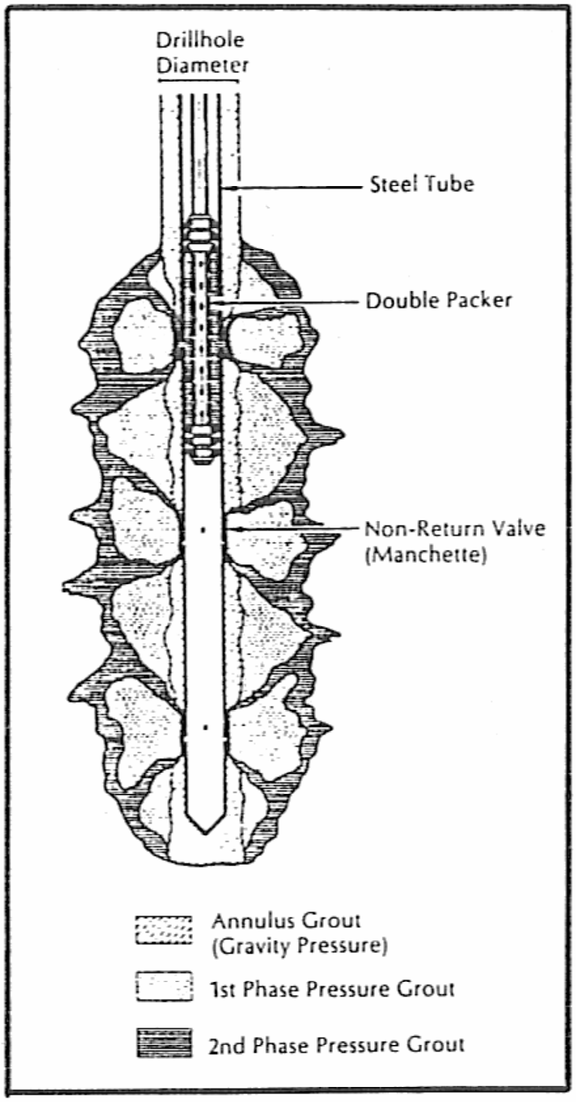

Figure 6. Concept of repeated post-grouting in increasing effective pile diameter. (Mascardi, 1982)

rubber sleeved ports at regular intervals (see Figure 6). Post-grouting greatly improves the grout/soil bond, but in addition it may increase the nominal cross section, particularly in the weaker soil layers or near ground level where natural in-situ horizontal stresses are small. Grout pressures of up to 300 psi are commonly used.

Mascardi (1982) noted that in cases of repeated post-grouting, an effective pile diameter in the range 12 to 30" may be expected, considering that standard pin pile construction normally provides bond zone diameters significantly larger than the nominal drill diameter. In general, pressure grouting is most effective in improving pile capacity in ground where deformations can be imparted relatively quickly: sands and gravels, residual soils, shales, and some weaker sedimentary and low grade metamorphic formations. Jones and Turner (1980) also noted that there was a very favorable response to post-grouting in stiff clay. No experience of good behavior in very soft non-consolidated clay or soft peat has been recorded.

Reinforcement of at least the upper part of the pin pile to guard against buckling or bursting is common and this concept has been exploited in most Nicholson type designs. In this case, the drill casing is pushed for some distance back down into the bond zone after the completion of pressure grouting, and is then left in place fully to the surface. This method provides excellent corrosion protection, eliminates the possibility of upper layer pile structural failure, and prevents the wasteful travel of grout into often very permeable upper horizons. The load/settlement performance is also superior, as illustrated in Figure 7, while additional resistance is automatically provided against lateral loading.

Pin piles furnish excellent load/settlement performance, with total deflections in normal cases being less than 1/2" at working load. However, there are cases where even this magnitude of movement is unacceptable to a particularly delicate structure. Preloading can then be used, wherein the pile has the working load preapplied via prestressing methods that induce settlement of the pile. Preloading can be accomplished in many ways. In the ROPRESS system, the pile is not bonded directly to the structure, but via an anchor pipe and screw. It is preloaded by a hydraulic jack, acting on the anchor pipe and locked off against the pipe when the desired load/deformation is achieved. An alternative system to preload pin piles has been used, achieving preloading by a strand anchorage founded below the pile tip. An additional benefit of preloading is that, as in ground anchors, each pile is routinely tested to at least its working load.

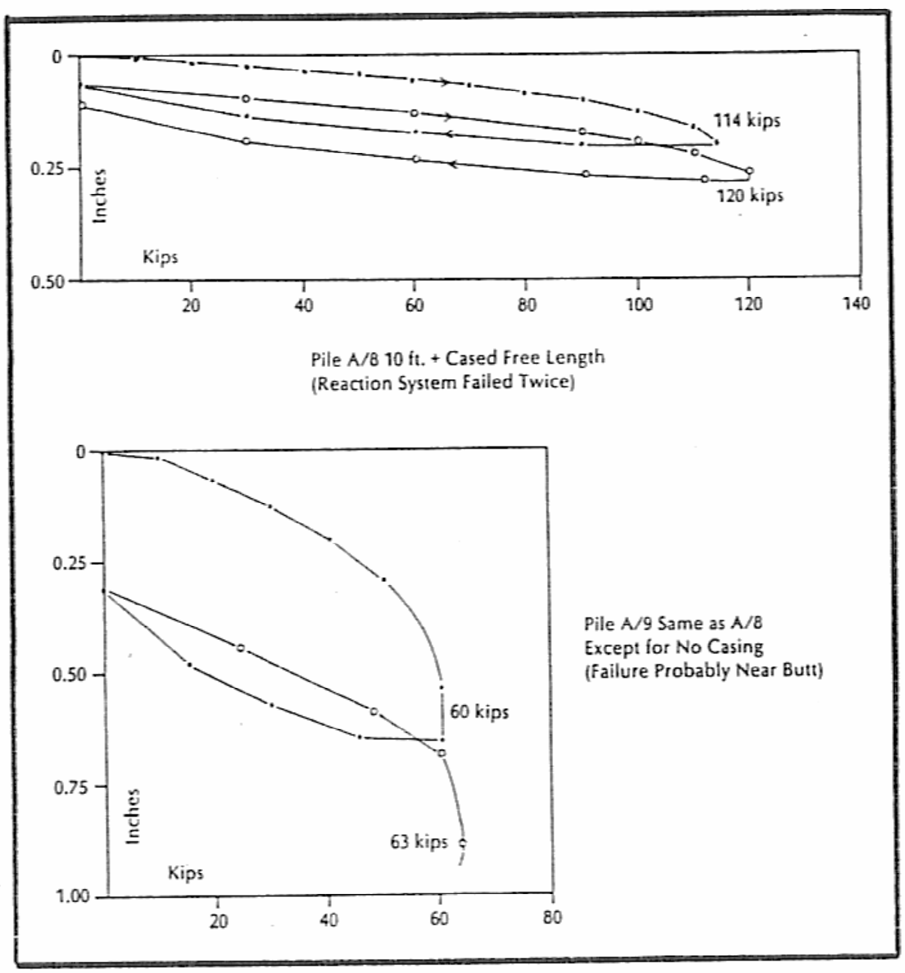

Figure 7. Pin pile load test data from a site in Coney Island, New York, showing the benefits of a steel liner through upper strata above the bond zone. (Bruce, 1988)

5. BENEFITS AND LIMITATIONS

The technique of pin piling is especially valuable in conditions where the ground is very variable and "difficult," where access is restrictive and where environmental considerations are highly significant, especially relating to vibration. Pin piles can be installed in almost any direction and through any structure or soil, and in close proximity to existing buildings. They can sustain extremely high loads relative to their diameter at exceptionally low deformations and can be installed so as to underpin structures with no settlement via preloading. Compressive, tensile and axial loadings can be accepted.

In most soil conditions, working loads of up to 50 to 60 tons can be generated safely, with far higher individual capacities recently recorded when founded in rock (Johnson and Schoenwolf, 1987). For very heavily loaded structures (e.g., bridge piers), groups of pin piles are, therefore, required. Even here, however, it would seem that the resulting group effect is positive, as opposed to demanding reduced individual design loadings.

Testing is relatively cheap, and by using adjacent piles as reaction, need not involve massive test frames and deadweights. There are a great deal of data available on pin pile performance in all ground types, and this has also undoubtedly helped the growth and acceptability of the technique.

The main limitation is cost - lineal costs are far greater than driven piles, for example. Nevertheless, circumstances often conspire, especially in urban construction, to make pin piling the only viable method of positive underpinned support.

6. TWO CASE HISTORIES

6.1. Boylston St., Boston, MA

6.1.1. 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 stories 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 (Figure 8)

Piling had to be executed from within the partially demolished basement of the structure (approximate floor elevation +8'), about 10' below existing sidewalk elevation giving a minimum headroom of 8'. The owner accepted an alternate bid based on the installation of about 260 pin piles of working loads 40 tons in compression, 12 tons in tension.

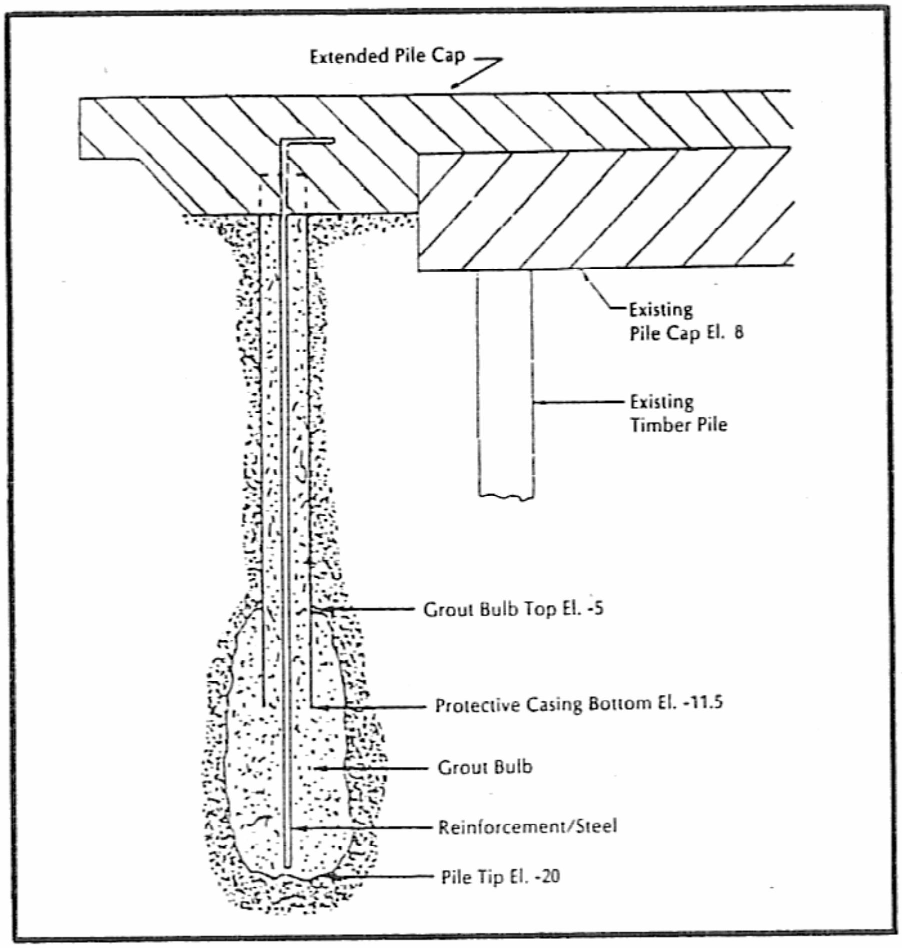

Figure 8. General arrangement of pin piles. Boylston Street. Boston. (Bruce, 1988)

6.1.2. 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.

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 18-24' 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.

6.1.3. Design

Piles were designed on the basis of an ultimate load 2.3 times design working load (i.e. 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 $\phi = 35^{\circ}$ for the sand, and a bulb diameter of 7-1/2" using a grout pressure of 60 psi in these soils.

Ultimate soil/grout bond (τult) was estimated empirically (Littlejohn, 1980) from

$$\tau_{ult} = \text{grout pressure} \times \tan\phi$$
$$= 60 \times 0.7 = \underline{42 \text{ psi}}$$

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

$$L = \frac{\text{Load}}{\pi \, d \, \tau_{ult}} \quad \text{where d is the bulb diameter}$$

$$\text{i.e.} \quad L = \frac{92 \times 2000}{\pi \times 7.5 \times 42} = 186" \text{ i.e. } 15' \, 6"$$

Further routine calculations using the provisions of the Massachusetts Building Code (1984) demonstrated that

- the use of 5-1/2" 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.

6.1.4. Construction

A diesel hydraulic trackrig was used to install all the piles. The 5-1/2" 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 15-16' of bond zone. The casing was then pushed back down 6-8' into this pressure grouted zone and left in place.

Grout takes generally ranged from 2.5 - 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, whilst the installation of an additional two piles lifted the contract total to 262.

6.1.5. Testing and Performance

Prior to the production piling program. 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 5-1/2" 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 and ASTM D1143. The data are summarized in Table 2, while the performance of TP2 (in compression) is shown in Figure 9 together with that of a timber pile, for comparison.

	BUTT (inches)		TIP (inches)	
	TP-1	TP-2	TP-1	TP-2
Compression Test (to 80 tons)				
Gross Settlement	0.44	0.34	0.31	0.19
Net Settlement (Permanent)	0.25	0.16	0.25	0.16
Tension Test (to 24 tons)				
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 Pile Data on Test Piles (TP) 1 and 2. Boylston St., Boston, MA.

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" (Av 0.16") - 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).

6.2 WARREN COUNTY, N. J.

6.2.1. Background

The I-78 dual highway crosses the Delaware River between Pennsylvania and New Jersey (Warren Co.) on new seven span, multigirder bridges. Generally bridge 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 15-20' 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.

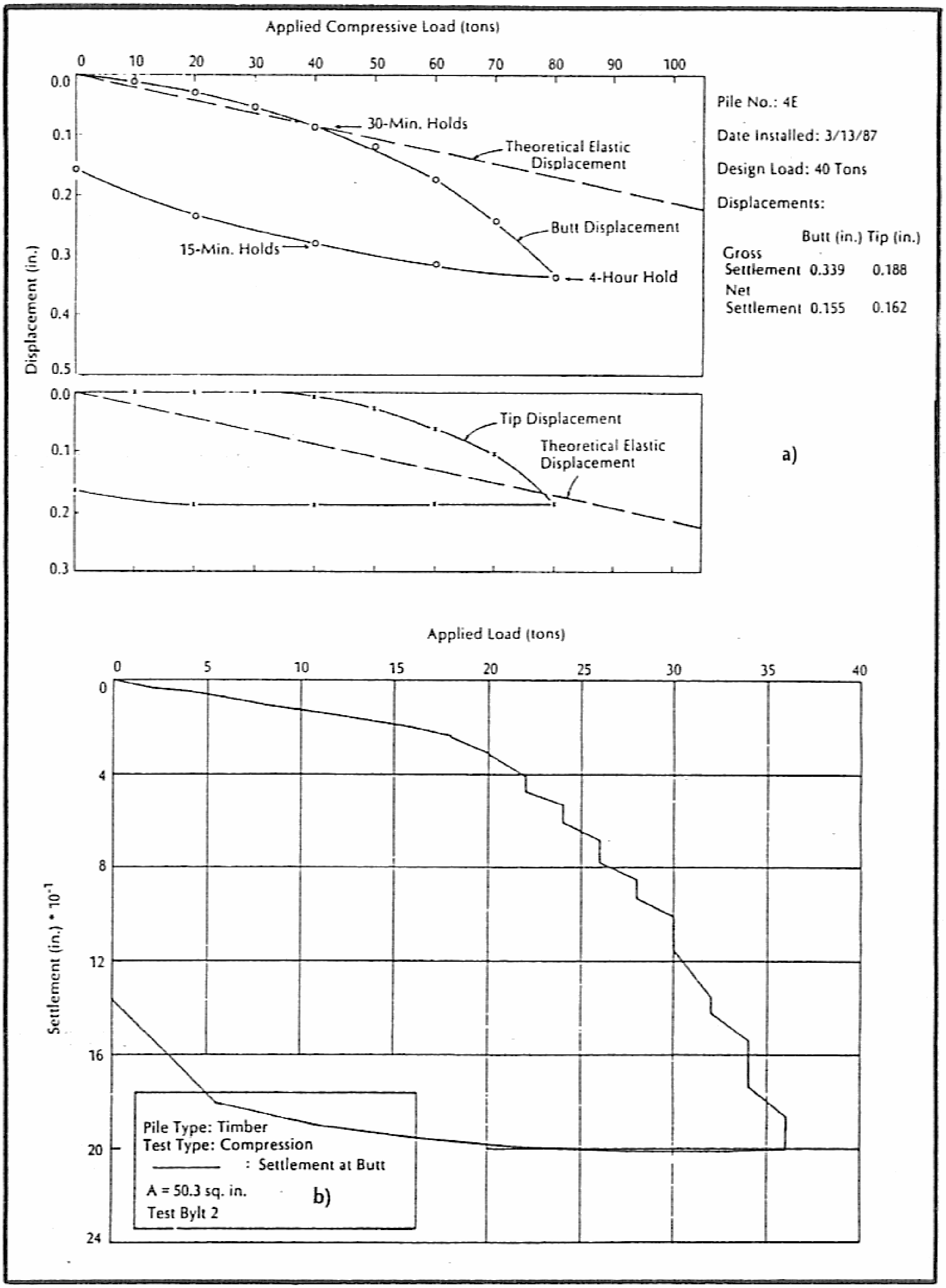

Figure 9. Load-settlement performance of: a) a drilled and grouted pin pile; b) a driven timber pile; Boylston Street. Boston.

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

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

The owner decided on the latter option on grounds of cost, program 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.

6.2.2. 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 e.g. 50' 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 Figure 10.

6.2.3. 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, 8-1/2" diameter and 15' long in competent rock.
- the use of a 55 ksi low alloy steel pipe of o. d. 7" and wall thickness 0.408" as pile reinforcement.

Recognizing 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 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

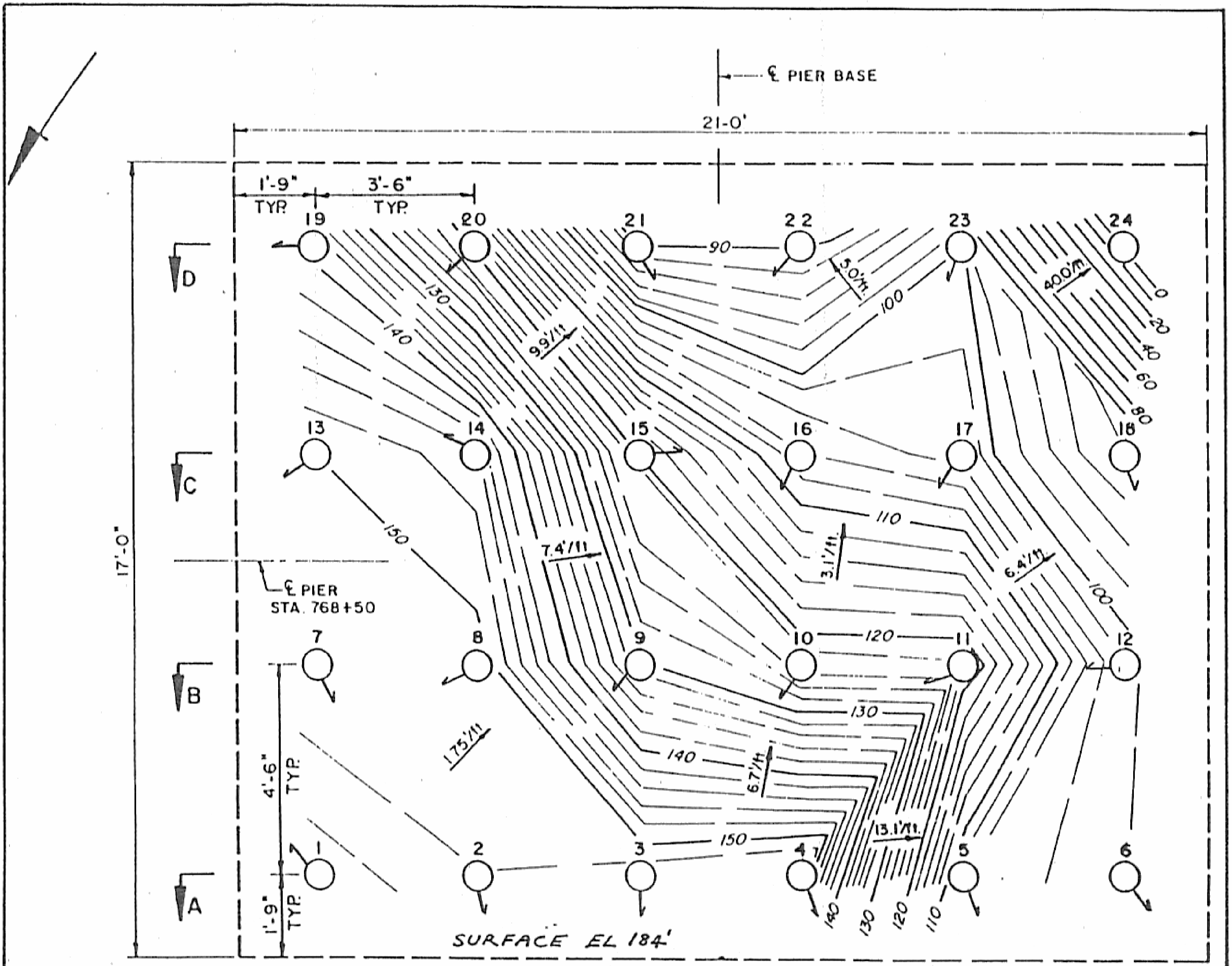

Figure 10. Interpreted bedrock isopachs, Warren County, NJ. Arrows show direction of drill hole deviaiton. (Data courtesy of Modjeski & Masters)

- 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.

6.2.4. Construction

The sequence of installation was as follows:

- o install 10.75" o. d. casing through the backfill and socket into the concrete of the cap.
- o drill with 10" down-the-hole hammer through the concrete footing.
- o install 9.625" casing through the less competent upper horizons (normally 30-45'). Survey linearity and grout in place.
- o drill 8.5" hole (by hammer or rotary) to ensure minimum of 15' bond zone as described above.
- o flush hole and install 7" o. d. reinforcing pipe. Survey for verticality (not more than 2% deviation allowable).
- o tremie grout hole pile and pressure to 50 psi.

Verification of each pile alignment was made through the use of an "R" Single Shot Direction Survey Instrument, manufactured by Eastman-Whipstock. Each pile was surveyed at top, bottom and mid depth. The results indicated that every pile fell within the criterion, with most being within 1% deviation.

Grout was mixed in a colloidal mixer and injected by Moyno pump. A neat Type III mix of w = 0.5 was used providing 3-day crushing strengths of over 3500 psi

Throughout construction, the very adverse geological 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. Figure 11 summarizes the actual total drilled lengths.

Regarding the anticipated caisson tip elevations, also shown in Figure 11, 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 lin. ft. corresponded with the total foreseen quantity of 1710 lin. ft. However, 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, with much of this being consumed in the zone above rockhead during pregrouting operations. The level of maximum takes corresponded with ground water level.

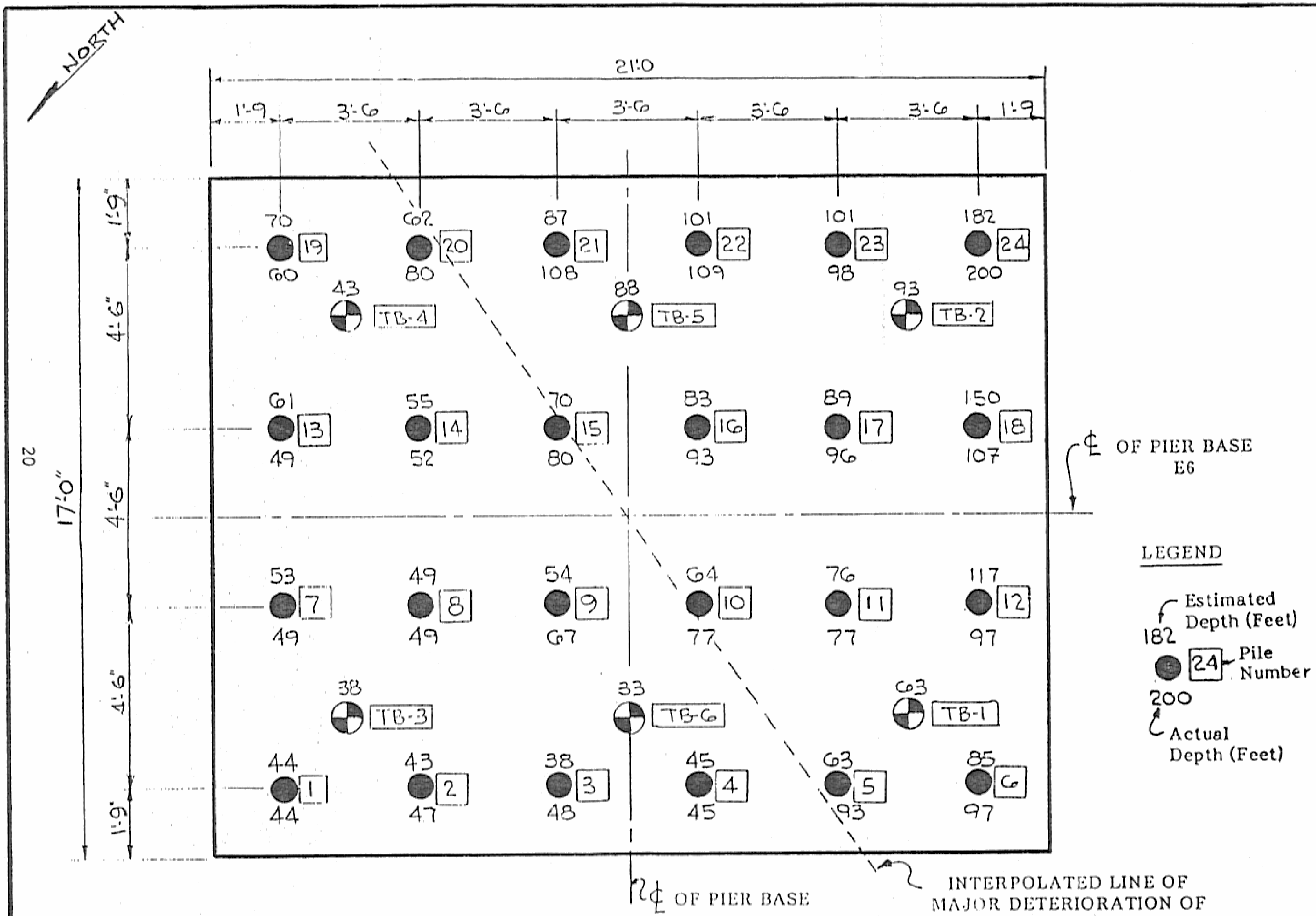

Figure 11. Actual pin pile lengths, with anticipated caisson depths shown for comparison. Warren County, NJ.

6.2.5. 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 205 tons, using rock anchors as reaction. This particular socket length was selected as at test load, the average grout-rock and grout-steel bonds would be 304 psi and 250 psi respectively - both considered to be at or near ultimate values. An outer sleeve of p. v. c. 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 total settlements recorded at each successive cycle to 205 tons 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 224 tons 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 of 0.371" at 215 tons, but 0.452" at 224 tons.)

In addition, during installation of the reinforcing pipe in the last (and deepest) pile (Nr24), 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 and recoupling, it was decided to grout the pile, having previously suspended a 20' long, 4-1/2" diameter 150 Ksi steel pin, with centralizers, 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 170 tons. The performance of the pile proved excellent, (e.g. total displacement 0.187" at 170 tons, 0.010" creep in 24 hours, permanent set of 0.009") and 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.

7. FINAL REMARKS

We hear a great deal about the tide of "European technologies" lapping at our shores. There is no doubt that pin piling was born in Europe, but it is equally certain that it is reaching a new maturity in this country. It has grown in response to the changing face of our construction industry and has developed its own particular national identity in the process: pin piles of exceptional length and capacity are being installed. There is an intensity of activity in certain of our older eastern cities unrivaled anywhere in the world. As redevelopment of our cities and industrial centers continues, we can expect the growth of pin piling to proceed apace, and the special national "flavor" to intensify.

ACKNOWLEDGEMENTS

At Boston, the owner was Pilgrim Management Trust. the engineer G.Z.A.. and the General Contractor, Perini Corporation. At Warren County, the owner was the

Pennsylvania DOT, the Engineer Modjeski & Masters, and the General Contractor GA and FE Wagman. The author appreciates the opportunity to describe details of these projects.

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