THE PRACTICE AND APPLICATION OF PIN PILING

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ABSTRACT: 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. Details from a major case history are reviewed. It is foreseen that the recent expansion in applications for pin piles in this country will continue apace in the light of the current construction trends, and the increasing appreciation of the great potential of the technique.

INTRODUCTION

It is now almost 40 years since the technique of minipiling, known widely as pinpiling in the United States, was first applied in Italy. Following the expiry of the original patents in the early 1970's, the popularity of the system spread rapidly throughout Western Europe and certain Far Eastern countries (8). This growth was stimulated by the trend of the construction industry in these areas: an emphasis on infrastructure development, redevelopment and upgrading in urban and industrial environments.

Pin piles provide exceptional load holding characteristics and can be constructed to considerable depths, through all types of ground and in very restricted access conditions. Their method of construction minimizes environmental impact and, in particular, the transmission of vibrations to soil and structure.

These advantages were, therefore, readily exploited in underpinning schemes to prevent or arrest structural settlements generated by the construction of adjacent excavations or tunnels, by changes in the ground water level, by increases in foundation loadings, or by the imposition of new machine vibrations to structures and foundations.

In the United States, the particular ramifications of undertaking complex ground engineering works in existing urban and industrial

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environments have impacted rather later (7). 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.

Thus, the major growth of pin piling in this country dates from the late 1970's, but has occurred at such a rate that there is now probably a greater intensity of pin piling in the cities of the East Coast than in any other region in the world.

Table 1 summarizes details from typical projects, and illustrates the wide range of pile applications, sizes, and capacities. 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 (5), or where they have acted as simple "pins" to stabilize the toes of sheet pile walls.

CONSTRUCTION

Certain authors (19) try to differentiate and classify pin piles in terms of diameter or load capacity. However, mode of construction is probably their most basic distinctive feature: all bored cast in place piles which can be installed with conventional drilling and grouting equipment may be referred to as pin piles. This definition practically limits diameter to 10 in. (254 mm) within the normal depths involved, i.e., 100 ft. (30m).

Construction details vary with the contractor and the project, but the sequence shown in Figure 1 is most common. The drill casing may be left in place above the load transfer horizon so as to improve performance under load and enhance corrosion resistance. Common features include:

- 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 ($0.7 MN/m^2$).
- Reinforcement: may be reinforcing cages (compressive loads only), high strength bars (compression or tension) or pipes (to resist bending stresses and permit very high loadings).
- 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 structural interface which is normally smooth 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 (2). Equipped with vibrating air driven pistons with tungsten carbide tips, the head is lowered into the (diamond drilled) hole and rotated slowly. The resulting roughened interface gives ultimate structure/ pile bond values up to ten times higher than conventional

FOUNDING STRATA	GEOMETRY Length (ft)	Dia (in)	COMPRESSIVE LOAD (tons) Working/Test	APPLICATION	MINIMUM HEADROOM (ft)	LOCATION	DISPLACEMENT (in) at Test Load Total	Permanent
PILES FOUNDED IN SOILS								
Dense sand, gravel with silt	30	5	10/20	New tank in existing waste-water treatment plant	18	Apollo, PA	0.049 0.077	0.008 0.022
Medium dense sand	30 to 55	5	55	Supporting masts of suspended net for new "natural" aviary	Swamp	Brookgreen Gardens, SC	Not Tested	
Sand and gravel	29	5	30/60	Existing dust collector structure on compacting soil	10-16	Neville Is, PA	0.078	0.010
Glacial till	62	5	27.5/55	Existing gymnasium building	20	Warwick, NY	0.188 0.249	0.002 0.005
Clayey sand and gravel	55 & 65	5	50/100	Existing operating coke battery, emission control facility	19-25	Monessen, PA	0.312	0.080
Dense sand and gravel	46 to 60	5 & 6-5/8	34 and 54	Existing corrosives storage tanks under which wood piles had failed	8-15	Mobile, AL	Not Tested	
Dense sand and gravel	30	5	50	Existing structure near deep excavation	Open Air	Pittsburgh, PA	Not Tested	
Dense sand and gravel	70	5	50/100	New emission control building at existing coke battery	25	Aliquippa, PA	0.200	0.020
Sand and gravels	23	5-1/2	10	New nuclear power structure in existing building	20	Apollo, PA	Not Tested	

(Continued)

FOUNDING STRATA	GEOMETRY Length (ft)	GEOMETRY Dia (in)	COMPRESSIVE LOAD (tons) Working/Test	APPLICATION	MINIMUM HEADROOM (ft)	LOCATION	DISPLACEMENT (in) at Test Load Total	DISPLACEMENT (in) at Test Load Permanent
Dense sand and gravel	75	5-1/2	50/100	Existing structure at Castle Building near deep excavation	Very restrictive access	Washington, D.C.	0.653	0.078
Medium dense sand	27	5-1/2	40/92	Redevelopment of existing building	8	Boston, MA	0.440 0.340	0.250 0.160
Medium dense sand	35 45	6-5/8 7-5/8	15/30 and 30/60	Rehabilitation of existing repair shop	8	Coney Island, NY	0.664 (uncased, 30 tons) 0.203 (cased, 57 tons)	0.317 0.006
PILES FOUNDED IN OR ON ROCK								
Sandstone	65 (8 ft. in rock)	6	55/110	Test to assess viability of underpinning existing granite sea wall	Open Air	Providence, RI	0.700	0.030
Weathered shale	36 (on rock)	5	10/20	New printing press in existing building	14	Trafford, PA	0.055	0.005
Sandstone, limestone	32 (3 ft. in rock)	4	10	Existing gantry runway	24	Burgettstown, PA	Not Tested	
Sandstone	26 (on rock)	5	45	Addition to water treatment plant	Open Air	Dunbar, PA	Not Tested	
Sandstone	43 (on rock)	5	55	Existing parking garage	8-10	Pittsburgh, PA	Not Tested	
Sandstone/shale	35 (on rock)	5-1/2	Various	New machine in existing building	20	Jeannette, PA	Not Tested	
"Bedrock"	70	7	60	Existing body stamping plant	18	Marian, IN	Not Tested	

(Continued)

FOUNDING STRATA	GEOMETRY Length (ft)	GEOMETRY Dia (in)	COMPRESSIVE LOAD (tons) Working/Test	APPLICATION	MINIMUM HEADROOM (ft)	LOCATION	DISPLACEMENT (in) at Test Load Total	DISPLACEMENT (in) at Test Load Permanent
Limestone	40 (1 ft. in rock)	5-1/2	70/140	New building in existing rolling mill	Open	Alcoa, TN	0.459	0.078
Sandstone	70 (on rock)	5-1/2	50	Restoration of existing timber court building	10	Pittsburgh, PA	Not Tested	
Karstic limestone	44 to 200 (15 ft. in rock)	8-1/2	100/224	New bridge pier	Open Air	Warren County, NJ	0.400	0.070
Limestone	35 (on rock)	5-1/2	40/80	New storage tank in existing building	11	Kingsport, TN	N/A	Zero
Sandstone/ siltstone	11.5 (all in rock)	6	45/68	Soldier beams for new retaining wall	Open Air	Pittsburgh, PA	0.059 to 0.099	0.006 to 0.020
Shale	142 (5 ft. in rock)	6-1/2	60	New addition to existing structure	Open Air	Cleveland, OH	Not Tested	

Table 1. Typical Pin Pile Projects (Conducted by Nicholson Construction Company)

Note: 1 foot (ft) = 0.3048 meters
1 inch (in) = 25.4 millimeters
1 ton (t) = 8.897 kiloNewtons

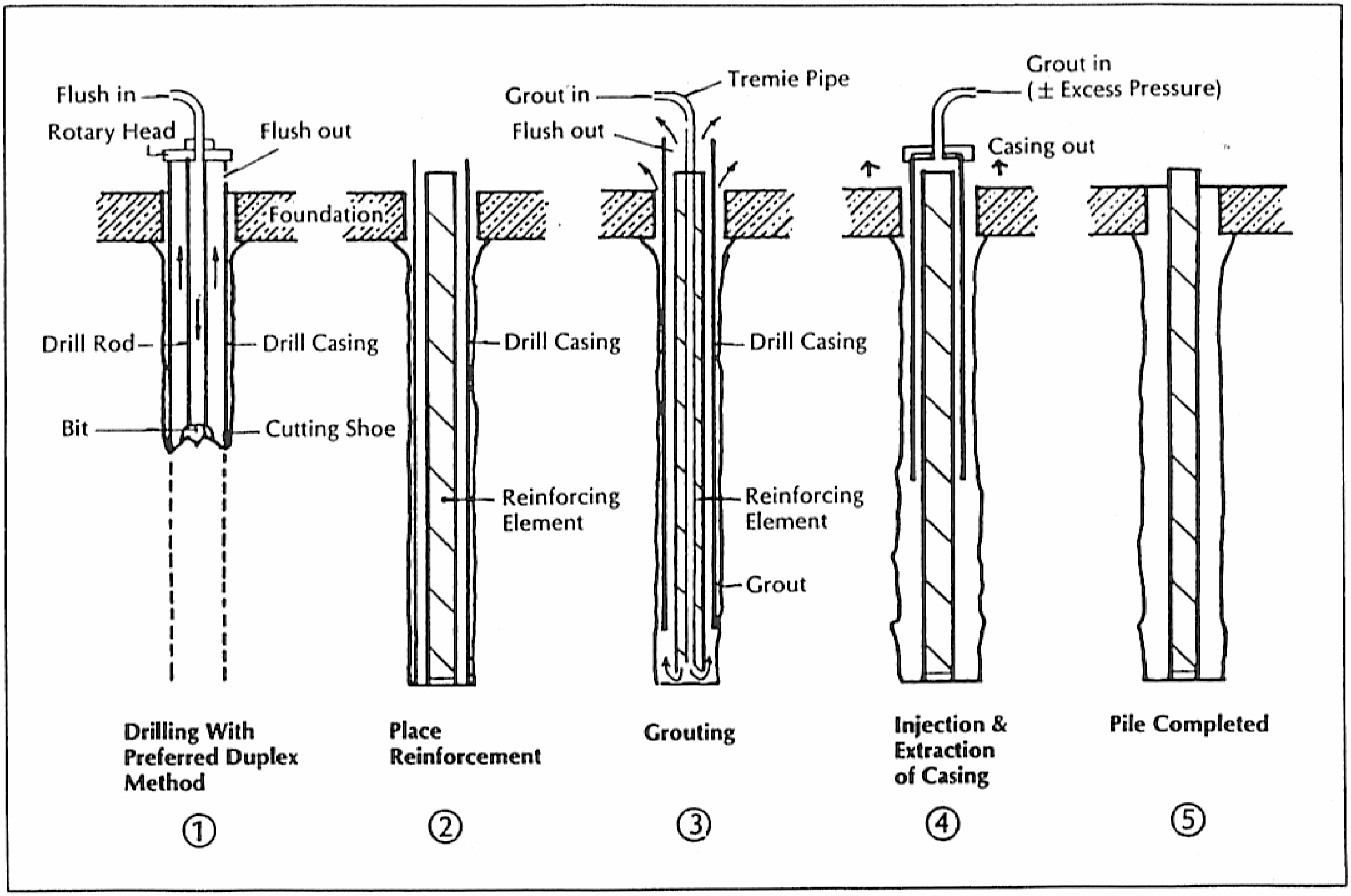

Figure 1. Stages in the construction of a standard pin pile (adapted from (12)). In stages 4 and 5 the casing may be left in place above the load transfer zone, to improve performance and corrosion resistance.

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 in. (25mm) is provided to the steel.

DESIGN AND PERFORMANCE

There are fundamentally two distinct design methodologies for pin piles, depending on their concept of operation: by skin friction or by end bearing. The former method is far more common. Pin piles are typically very slender elements in which the lateral area is hundreds of times greater than the base and, of course, relative displacements needed to mobilize frictional resistance are much smaller than those necessary to develop end bearing. Therefore, the load is considered to be transferred by skin friction in the "socket" length in the bearing stratum, be it suitable soil, or soft-medium rock, and is usually done so at surprisingly small pile settlements. In contrast, pin piles are occasionally conceived to act as struts - transferring structural loads through incompetent upper horizons (e.g., fill or soft clay) directly onto an underlying hard bedrock surface. In such cases no "socket" length is drilled and the load is transferred to the rock only across the pile's base. The calculation of pile geometry and composition in this case is a straightforward matter once allowable end bearing pressures and material safety factors have been set.

Referring back to the more common case, the method of 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 (Figure 2), albeit to interfaces in the opposite sense of shear. As a general guide to the design of the transfer length, the PTI Recommendations (18) may be followed. These provide detailed guidance for the determination of grout to soil skin friction values for different ground conditions and grouting methods.

With these major points in mind, the basic design philosophy does not differ from that of 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 viewpoint, corrosion resistance and compatibility with the existing ground and structure (during construction) must be regarded. The system must also be economically viable. Reference must always be made to local construction regulations for guidance, although the special aspects of pin piles are not often adequately or specifically addressed. In that event, sensible interpretation is necessary.

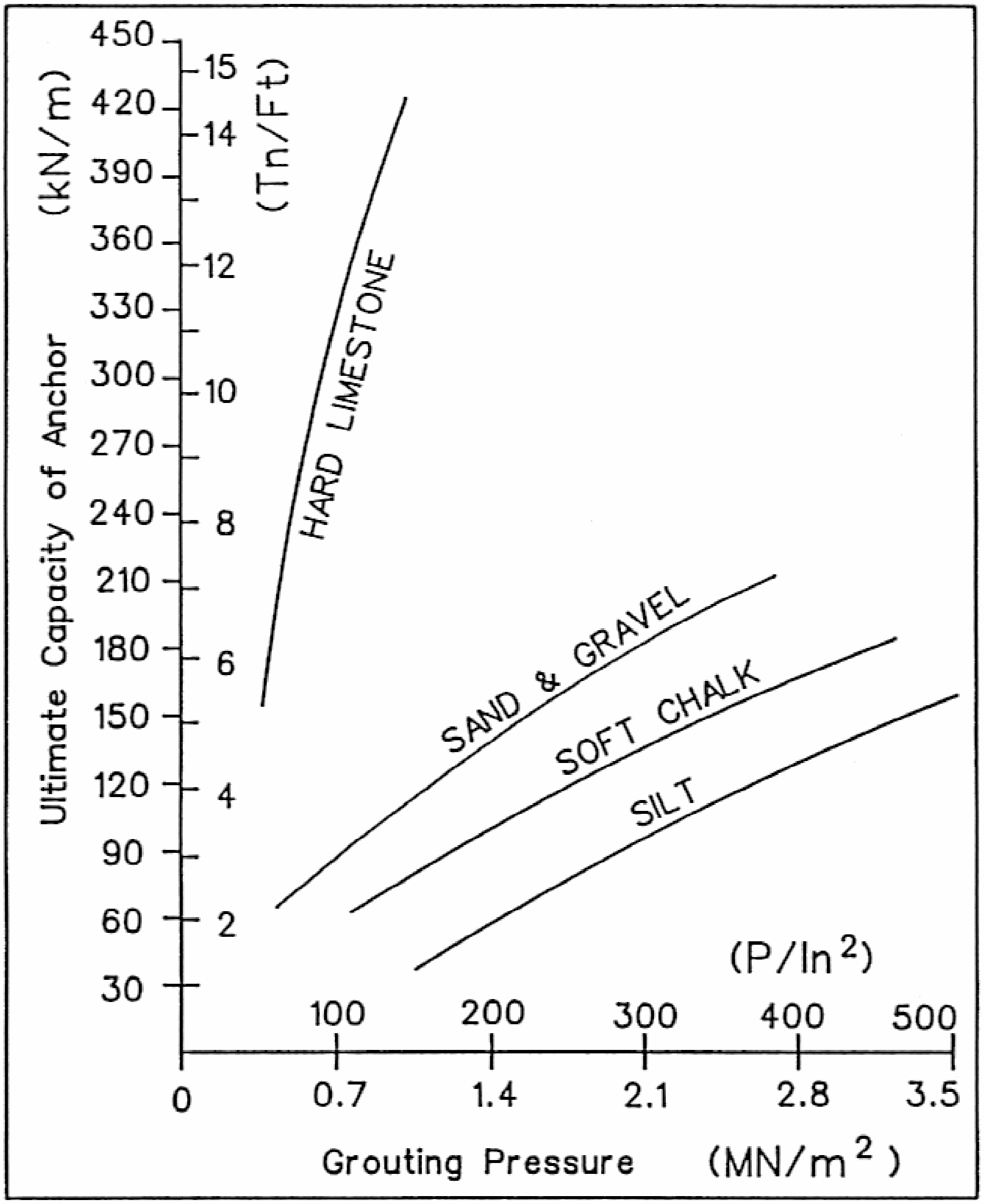

Figure 2. Influence of grouting pressure on ultimate load holding capacity (13).

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 strength parameters, and not just 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 (4) is supported by the detailed analyses of Mascardi (15, 16) and Gouvenot (9). These authors conclude that only in soils of the very poorest mechanical properties (such as loose silts, peat and non-consolidated clays) where the value of the elastic modulus is less than 70 psi ($0.5 MN/m^2$), 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, adding reinforcement or by maintaining a permanent casing through dubious horizons.

A final point may be made in relation to the "group effect" - a common consideration in pin pile design. The contrast with conventional piling is fundamental. For example, the British Code of Practice 2004 (6) states that for "friction piles, the spacing center-to-center should be not less than the perimeter of the pile; with piles deriving their resistance mainly from end bearing, the spacing center-to-center 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, a "knot effect" (14, 3, 17) has been reported whereby a "positive" group influence has been achieved during loading of the pile-soil system. This is enhanced by inclining the piles within any one group in different directions. While the precise degree of improvement is difficult to quantify, as it is dependent on so many factors, including the soil conditions themselves, it may be concluded that the close spacing of pin piles certainly does not adversely affect individual pile capacity.

DEVELOPMENT TRENDS

In the last decade, as demand has increased and the number of specialist contractors qualified to do the work has grown, several innovations have been made in aspects of pin piling. These have been directed towards providing piles of superior performance more economically and in more challenging structural or environmental settings. Three developments merit particular attention:

- Post-grouting of the bond zone
- Reinforcement of the 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 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 type piles, for example, wherein the packer is introduced into the steel core pipe and grout is ejected through the rubber sleeved ports at regular intervals as shown in Figure 3. Post-grouting greatly improves the grout/soil bond but, in addition, it may increase the nominal pile cross section, particularly in the weaker soil layers. Grout pressures of up to 300 psi (2.1MN/m^2) are commonly used.

Mascardi (16) noted that in cases of repeated post-grouting, an effective pile diameter in the range 12-30 in. (300-760mm) 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 conditions where deformations can be imparted relatively quickly: sands and gravels, residual soils,

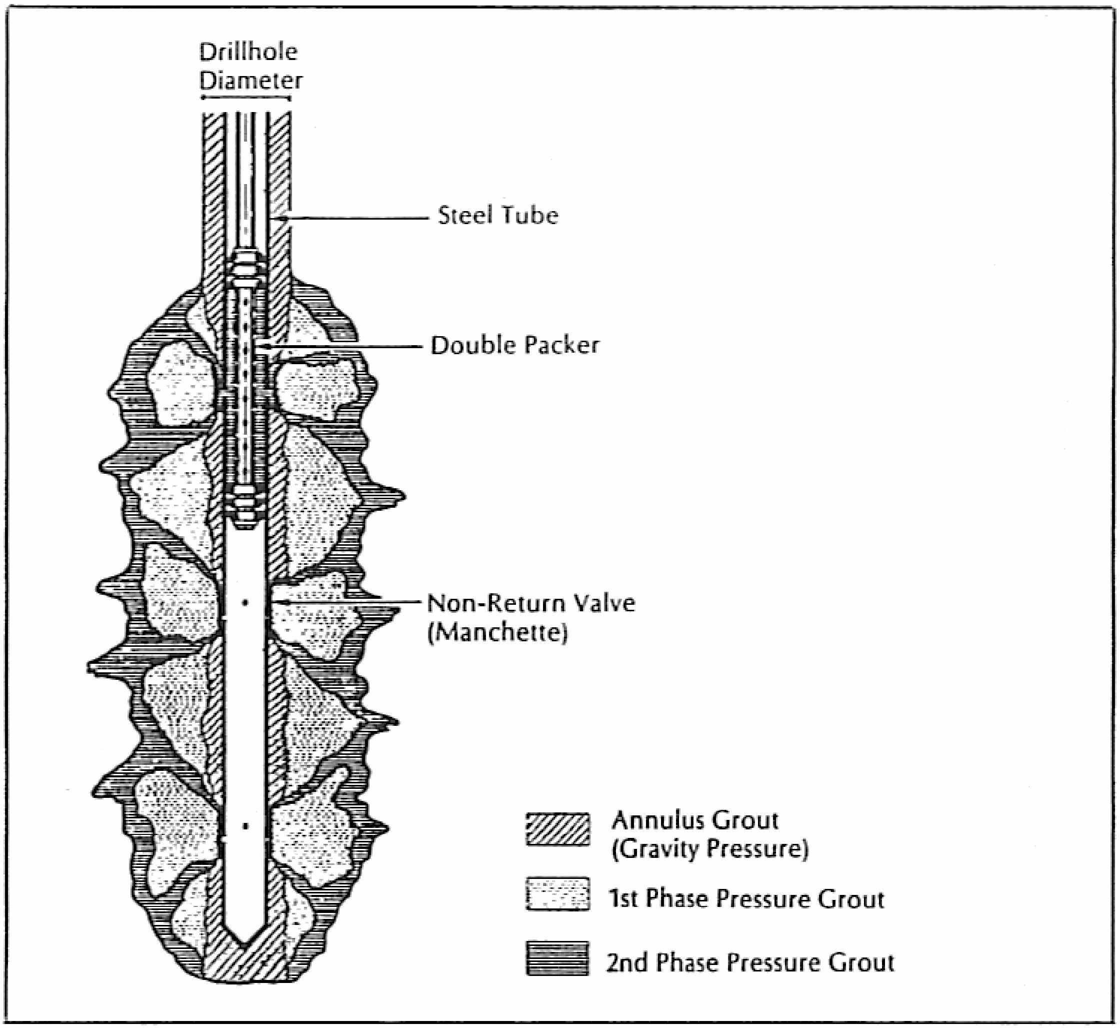

Figure 3. Concept of repeated post grouting in increasing effective pile diameter (16).

shales, and some weaker sedimentary and low grade metamorphic formations. Jones and Turner (11) also noted that there was a very favorable response to post-grouting in stiff clay. There appear to be no examples of pin piles founded in very soft clays or organic materials.

Reinforcement of at least the upper part of the pin pile to guard against buckling or bursting has become common and standard practice for certain contractors. As shown in Figure 1, the drill casing is pushed for some distance back down into the bond zone after the completion of pressure grouting. It is thus left in place from that depth, to the surface. This method provides excellent corrosion protection, eliminates the possibility of pile failure in upper horizons and prevent the wasteful travel of grout into the same horizons which are often loose and permeable. The subsequent load/settlement characteristics are also superior, as illustrated in Figure 4, while additional resistance is provided automatically against lateral loadings.

The performance of pin piles under load is generally excellent, with total deflections for pin piles of average geometries normally being less than 1/2 in. (12mm) at working load. However, there are cases when even this magnitude of movement is unacceptable to a particularly sensitive structure. Preloading can then be used,

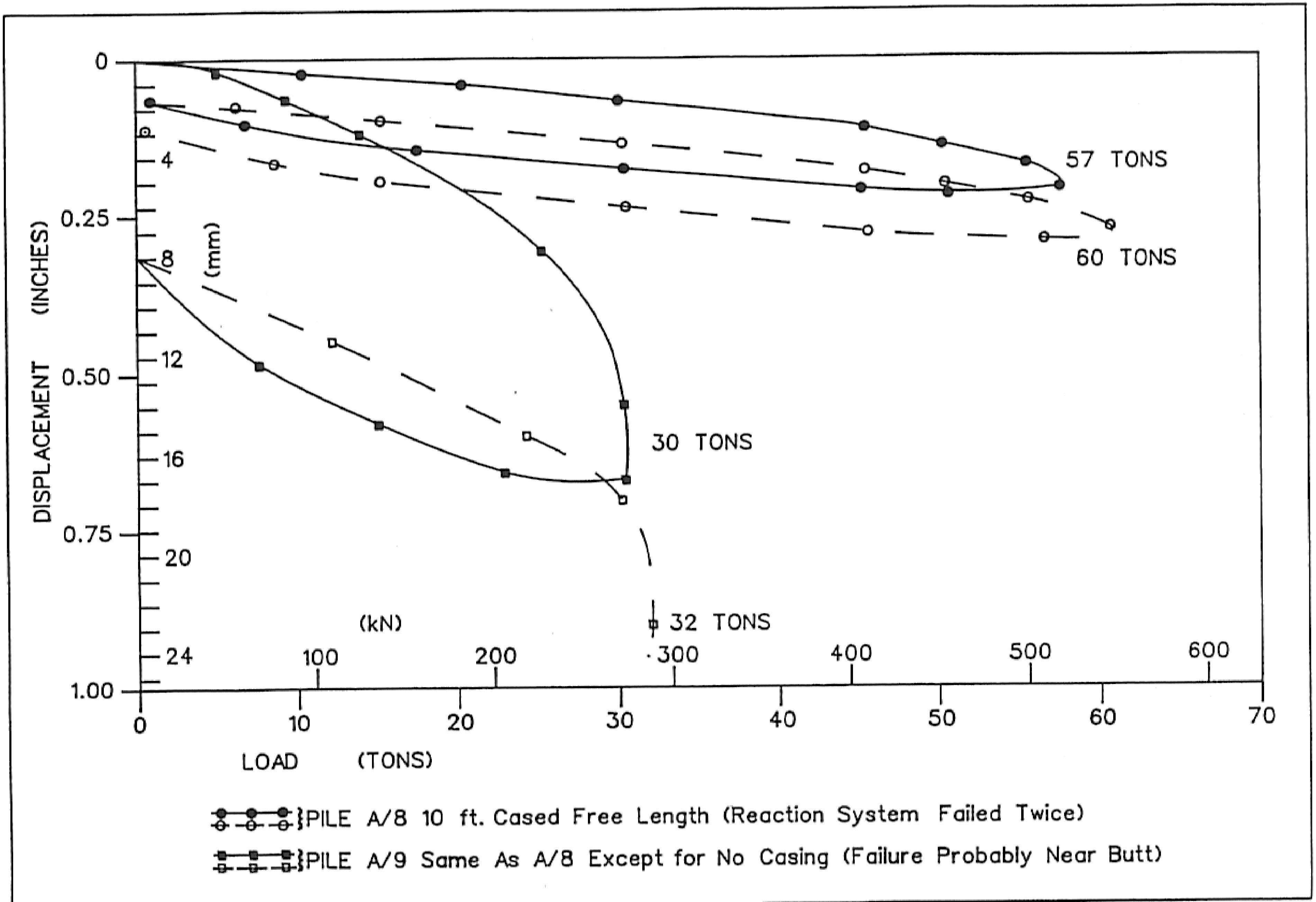

Figure 4. Pin pile load test data, from a site in Coney Island, NY, showing the benefits of leaving a steel casing through the upper strata (7).

wherein the pile has the working load preapplied via prestressing methods that induce settlements prior to the pile's connection to the structure. Preloading can be accomplished in several ways but the most common is illustrated by the project at Warwick, New York, referred to in Table 1. Two 0.6 in. (15mm) diameter prestressing strands were installed coaxially through the pile and anchored 20 ft. (6m) into the soil below the pile tip. After the grout had reached target strength, the strands were stressed by a hydraulic jack, placed on the pile cap and so reacting down on the pile itself. The equivalent of pile working load was applied, thereby inducing pile cap settlement. This load was afterwards released as the final structural load was in due course transferred from the building being refurbished. Readings at 24 locations on the structure subsequently showed a maximum of 0.01 in. (0.3mm) of additional structural settlements during and after reconstruction, compared with the 0.25 in. (6mm) limit set.

An additional benefit of preloading is that, as in the case of prestressed ground anchorages, each installation is routinely tested to at least its working load.

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 - especially the transmission of vibrations - are highly significant. 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 surprisingly high loads relative to their diameter at exceptionally low settlements and can be installed with preloading facilities to underpin very sensitive structures with no additional settlement. Both compressive and tensile axial loadings can be sustained, while lateral forces can be accomodated with appropriate design and construction methods.

In most soil conditions, working loads of up to 60 tons can be generated safely using standard construction methods, while working loads around 100 tons can be provided using newer grouting techniques. When founded in rock, certain pin piles have been installed at recent projects (10) which have been tested to loads approaching 500 tons. Generally, highly loaded structures, such as bridge piers, require groups of piles. However, even in this situation it would seem that the resulting "group effect" is positive although further field research is necessary to quantify the influence of geometry and geology.

Testing is relatively economical and by exploiting adjacent piles as reaction, need not involve massive test frames and dead weights. There are a great deal of data available on pin pile performance in different ground conditions and the degree of confidence afforded by this body of information has aided the growth and acceptability of the technique (7).

The main limitation is financial. Lineal costs are somewhat 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.

Background. The I-78 dual highway crosses the Delaware River between Pennsylvania and Warren County in New Jersey 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 to 20 ft. (4.5 to 6.0m) 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.

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 ft. or 0.9m) caissons each of working load 360 tons
- 24 mini piles, each of nominal working load 100 tons (allowing an 11 percent 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 mini piles 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 is a Cambro-Ordovician dolomitic limestone referred to locally as the Allentown Limestone. At the site the rock was found to be moderately to highly fissured, cherty, and very susceptible to karstic weathering. Major clay filled beds were intersected even at depths over 100 ft. (33m) below the surface. For example, 50 ft. (15m) of soft brown silty clay was recorded below rock at 106 ft. (32m) during drilling at Pile 24 (Figure 5). The mass was as variable laterally as it was vertically and its rockhead varied from 32 to 184 ft. (10 to 55m) beneath the surface. The regional dip is 55^{0} to the southeast.

Design. The owner's design regulations permitted:

- maximum average rock-grout bond of 50 psi ($0.3 MN/m^2$) at the working load of 100 tons.
- maximum allowable reinforcement steel stress ("fa") equivalent to 45 percent of fc, at the working load.

These factors led to the selection of:

- a load transfer zone, 8-1/2 in. (216mm) in diameter and 15 ft. (4.5m) long, in competent rock.
- the use of a 55 ksi ($380 MN/m^2$) low alloy steel pipe of

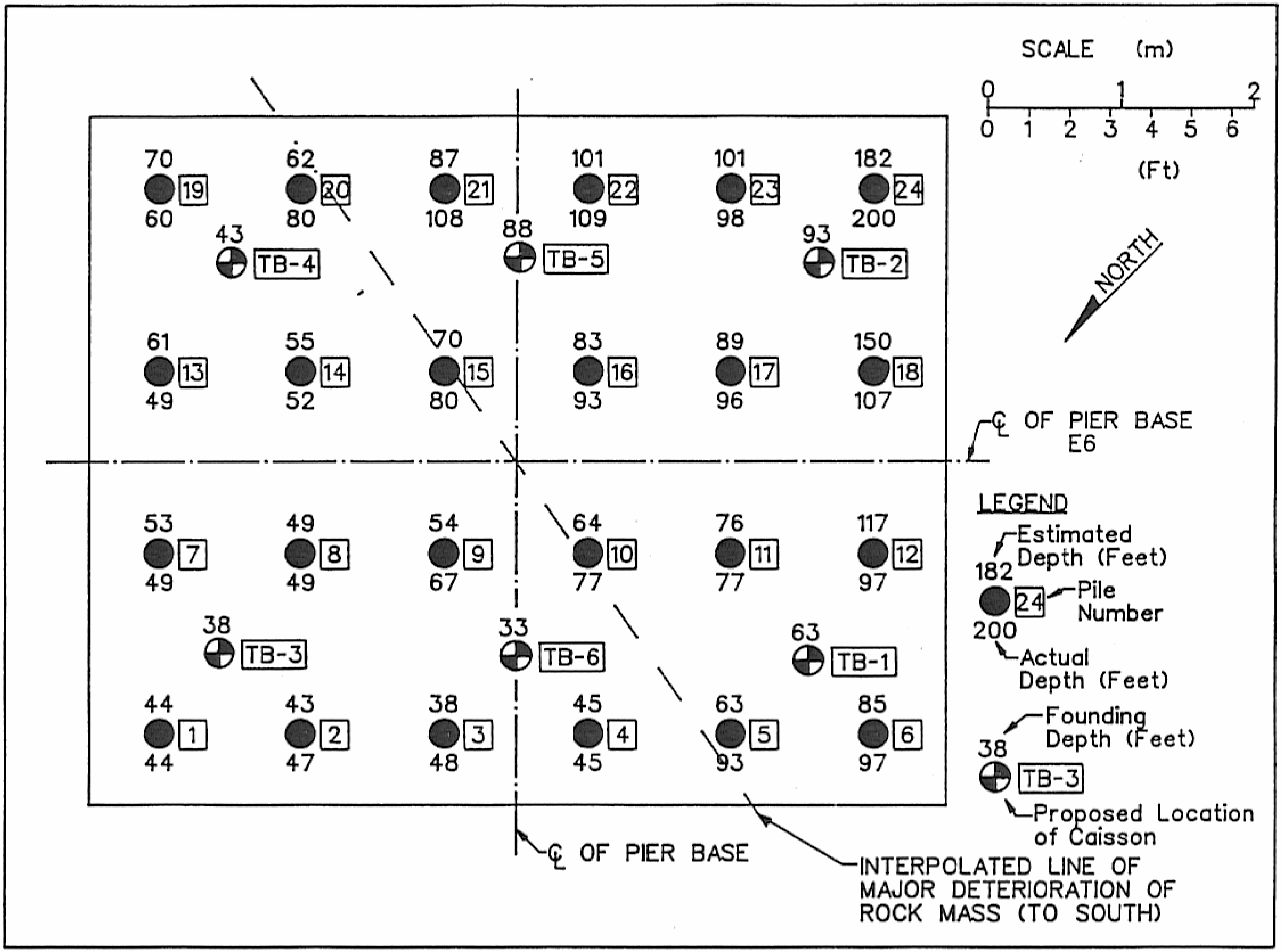

Figure 5. Actual pin pile lengths and foreseen caisson depths.
Pier E6, I-78 Bridge, Warren County, NJ.

outside diameter 7 in. (178mm), and wall thickness 0.408 in. (10mm) as pile reinforcement.

Recognizing that rock conditions were likely to be very variable, provision was made to allow the load transfer zone to not necessarily be continuous, in all piles except numbers 1, 6, 17, 18, 19, 23, and 24, but subject to the following restrictions:

- the lower part of the zone to contain at least 10 ft. (3m) of continuous sound rock
- soft interbeds to be less than 3 ft. (0.9m) thick.
- a zone of acceptable load bearing rock to be at least 5 ft. (1.5m) thick.
- regrouting and redrilling of interbeds within the overall bond zone to be undertaken.

<u>Construction.</u> The sequence of installation was as follows:

- install 10.75 in. (273mm) outside diameter casing through the backfill and socket into the cap concrete.
- drill with a 10 in. (254mm) down the hole hammer through the concrete footing.
- install 9.625 in. (244mm) casing through the less competent upper horizons, survey its verticality and grout in place.
- drill on at 8.5 in. (216mm) diameter to provide minimum of 15 ft. (4.5m) of bond zone, as described above.
- flush the hole and install 7 in. (178mm) reinforcing pipe.
- survey for verticality (limit of 2 percent deviation permissable).
- tremie grout into hole and pressure to 50 psi (0.3MN/m^2).

Verification of pile verticality was made by use of a Single Shot Direction Survey Instrument. Each pile was surveyed at top, bottom, and mid-depth. The results indicated that every pile bettered the criterion, with most having less than 1 percent deviation.

Grout was mixed in a colloidal mixer and injected with a rotary progressive cavity pump. A neat Type III cement grout mix was used, with a water/cement ratio of 0.5 by weight. Three day crushing strengths exceeded 3500 psi (24 MN/m2).

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 5 summarizes the actual total drilled lengths.

Regarding the anticipated caisson tip elevations, also shown in Figure 5, these would in all cases have been shorter than subsequently proved necessary to found safely the pin piles. Poor or voided rock was found consistently below these anticipated elevations, further supporting the decision to use the small diameter elements.

The total actual drilled length of 1420 ft. (585m) was 14 percent greater than the total anticipated amount. However, within this total variations from 43 ft. (13m) less to 30 ft. (9m) more were recorded on individual piles high-lighting the local variability of the rock. Overall, a volume of grout equivalent to four times the nominal hole volume was injected, with much of this being consumed in the zone above rockhead during pregrouting operations. The level of maximum takes corresponded to the piezometric surface.

Testing and Performance. A separate test pile 30 ft. (9m) long with only 5.33 ft. (1.6m) of bond was load tested in accordance with ASTM D1143 "QuickLoad Test Method" (1) to 205 tons, using rock anchors as reaction. This short bond zone was chosen so that, at test load, the average grout - rock and grout steel bond values would be 304 psi (2.1MN/m^2) and 250 psi (1.7MN/m^2) respectively - each considered to be very close to ultimate values. An outer plastic sleeve extending to the top of the bond zone ensured load transfer only in that zone. A 6 in. (152mm) thick, soft wooden plug was attached to the bottom of the reinforcing pipe prior to installation, to ensure that no load would be transferred in end bearing.

The load displacement data are shown in Figure 6. The performance in the first two cycles to 205 tons was excellent in terms of permanent set and creep, both of which were minimal. During the third cycle, the upper pile materials began to fail at 224 tons. To that point the pile had behaved as previously, with the total displacement at 215 tons being 0.371 in. (9.4mm), compared to 0.452 in. (11.5mm) at 224 tons.

In addition, during installation of the reinforcement into the last - and deepest - hole, drilled for Pile 24, a thread parted and a 130 ft. (40m) length of pipe fell into the 200 ft. (61m) deep hole. A borehole TV camera revealed that the pipe had been further ruptured about 30 ft. (9m) above the base, due to the impact. After several abortive attempts at recovery and decoupling, it was decided to grout the pile, as normal, but after first suspending a centralized steel pin from 62 ft. (19m) to 82 ft. (25m) below the surface. This 20 ft. (6m) long pin, 4-1/2 in. (114mm) in diameter,and consisting of 150 ksi (1034 MN/m^2) steel was intended to ensure effective load transfer across the upper discontinuity. A very rigorous extended load test was then conducted to a maximum of 170 tons. The performance of the pile proved excellent, with a total displacement of 0.187 in. (4.7mm), and 0.01 in. (0.3mm) creep after 24 hours at maximum load, followed by a permanent set of 0.009 in. (0.2mm). It was approved as being capable of performing safely its function in service.

It is understood that long term records of pier performance with time were not maintained during and after subsequent bridge deck construction. However, the bridge has been completed and is operating to the satisfaction of all parties. It may, therefore, be assumed that Pier E6, and its foundations are functioning at least as well as the other piers with their more conventional foundation systems.

CONCLUSIONS

There is much talk about the so-called 'new technologies" and their impact on American construction practices. There is no doubt that pin piling was born in Europe, but it is equally clear that it is reaching a new maturity in this country. It has grown in response to the changing trends in 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

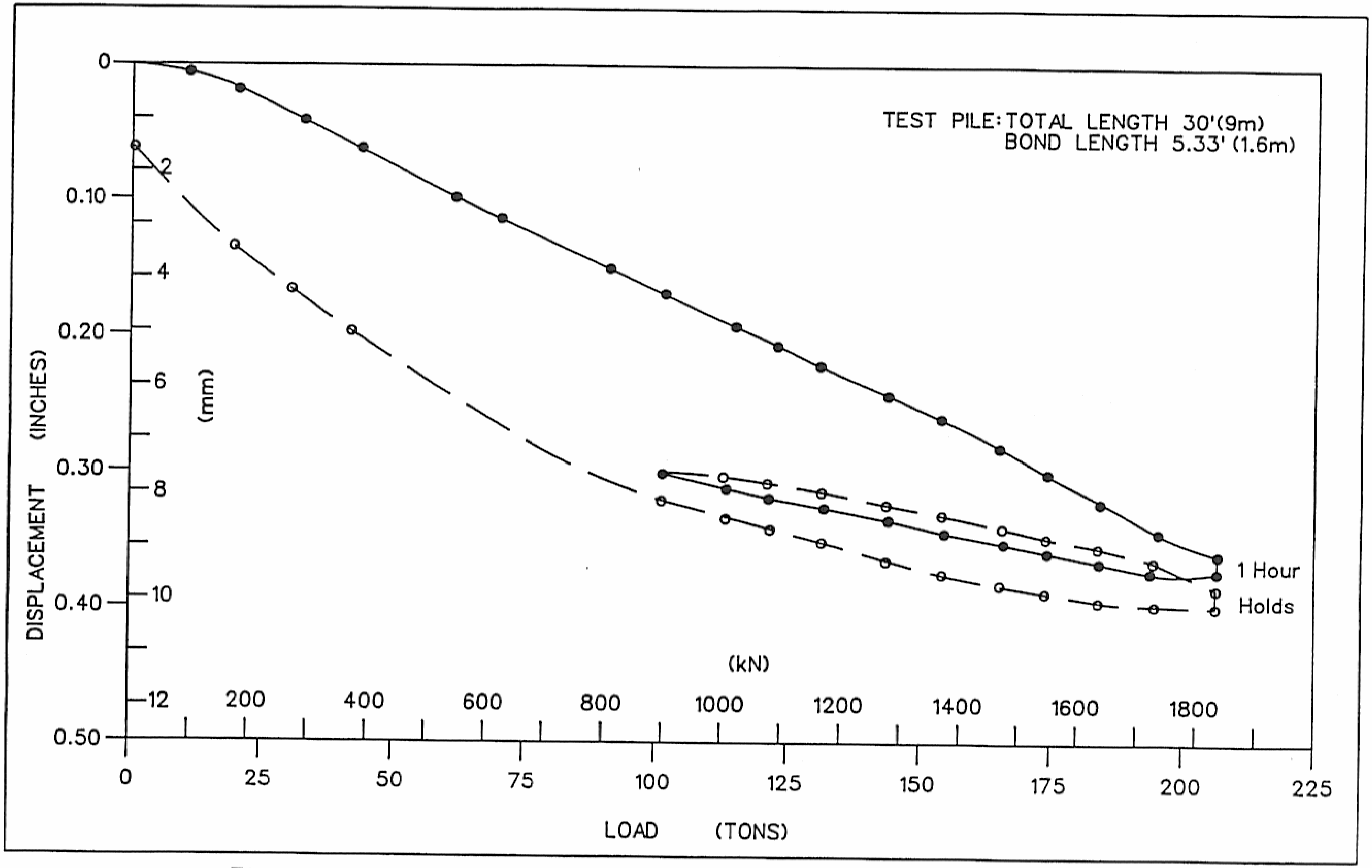

Figure 6. Load-displacement data, test pile at Pier E6, I-78 Bridge, Warren County, NJ.

is an intensity of activity in certain of our older eastern cities unrivalled anywhere in the world. As redevelopment of our cities and industrial centers continues, we can expect our distinctive national "flavor" to intensify.

ACKNOWLEDGEMENT

The owner of the I-78 bridge over the Delaware River is the Pennsylvania Department of Transportation. The Consulting Engineers were Modjeski and Masters, and the General Contractor, G.A. and F.E. Wagman. The authors appreciate the opportunity to describe details of this project.

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