THE USE OF SMALL DIAMETER INSERTS AS PILES AND INSITU REINFORCEMENT

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SUMMARY

The use of small diameter cast-in-place bored piles is becoming increasingly popular. Such inserts are used as conventional load bearing piles, and are nationally referred to as pin piles. The other major application is as the in situ reinforcement of soil slopes or excavations. The paper describes the applications, construction, design and performance of inserts of each type, and a case history from each category is provided.

1. PREAMBLE

In the last decade or so in this country, there has been an increasing use of small diameter cast-in-place bored inclusions. By far the greatest number have been installed to act as conventional piles to accept direct structural loadings and transfer them to deeper, more competent horizons. The balance, although normally installed in exactly the same way, have been intended to act as in situ reinforcement to maintain soil equilibrium under the soil and surcharge loading.

It is essential to differentiate between these two groupings, as their applications, design and performance are clearly completely distinct. And yet this fundamental subdivision is rarely if ever drawn, to the confusion of potential clients and to the loss of potential contractors. This situation was fostered by certain European specialist companies in the earlier days of the techniques, in their desire to maintain some type of proprietary mystique. In addition, tests have proved that with certain geometries, closely spaced piles can also generate the in situ reinforcing effect to improve load/settlement performance. It is the intention of this paper to review both grouping, and to clarify the issue in the interests of all concerned, bearing in mind the considerable potential of both systems in urban and highway engineering.

2. PILING

Such inclusions used as piles are referred to generically as mini piles, although in the United States the term pin piling is achieving national recognition. Pin piling is now well into its fourth decade of application. Comprehensive reviews of design, construction and performance are readily available (e.g. Koreck 1978, Weltman 1981, Lizzi 1982, Bruce et al 1985, and Bruce 1988), whilst a recent synthesis was provided by ASCE (1987).

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2.1. Evolution and Applications

Small-diameter cast-in-place bored piles saw their first application in Naples in 1952. With the expiration of the original patents in 1972, their application became much more widespread, fostered by the upsurge in remedial and subterranean works in urban areas, and the mass of supportive test data available from (cheap) load test programs. Acting singly or with the benefit of the group effect, load-bearing pin piles have been used in many applications wherein the transfer of structural loads to more competent lower horizons has had to have been achieved with minimal environmental disturbance or structural settlement.

The applications basically revolve around the prevention or arrest of structural settlements generated by adjacent underground excavation, changes in the groundwater level, changes to foundation loadings (by additions) or the imposition of machine vibrations to structures and foundations.

In Boston, a typical example was the recent support of the addition to the Dana Farber Cancer Institute where a total of 165 piles with a 75-ton working load were installed in a sub-basement having 3m of headroom through the clay and into bedrock. Also in Boston, the Hynes Auditorium renovation and expansion required the installation of over 330 piles with up to 250-ton working loads (Johnson and Schoenwolf, 1987). Over half of the piles were installed with headroom as low as 3m. This project remains the largest single contract, in dollar terms, yet executed in the U.S. In Brooklyn, over 4,000 piles of 15- to 30-ton working load are currently being installed in the Coney Island Main Repair Facility of the New York Transit Authority with minimum disruption to shop operation (Munfakh and Soliman, 1987). In this case, the piles penetrate loose fill and swampy, peaty deposits before being founded at depths of 10 to 13m in dense fine-medium sands. They support the floor slab and its upgraded live and dead loads.

2.2. 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 try to differentiate pile nomenclature in terms of diameter (Weltman, 1981), 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 25 cm for piles within the normal depths involved (less than 25m).

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 0.8 to 1.0 N/mm^2.

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

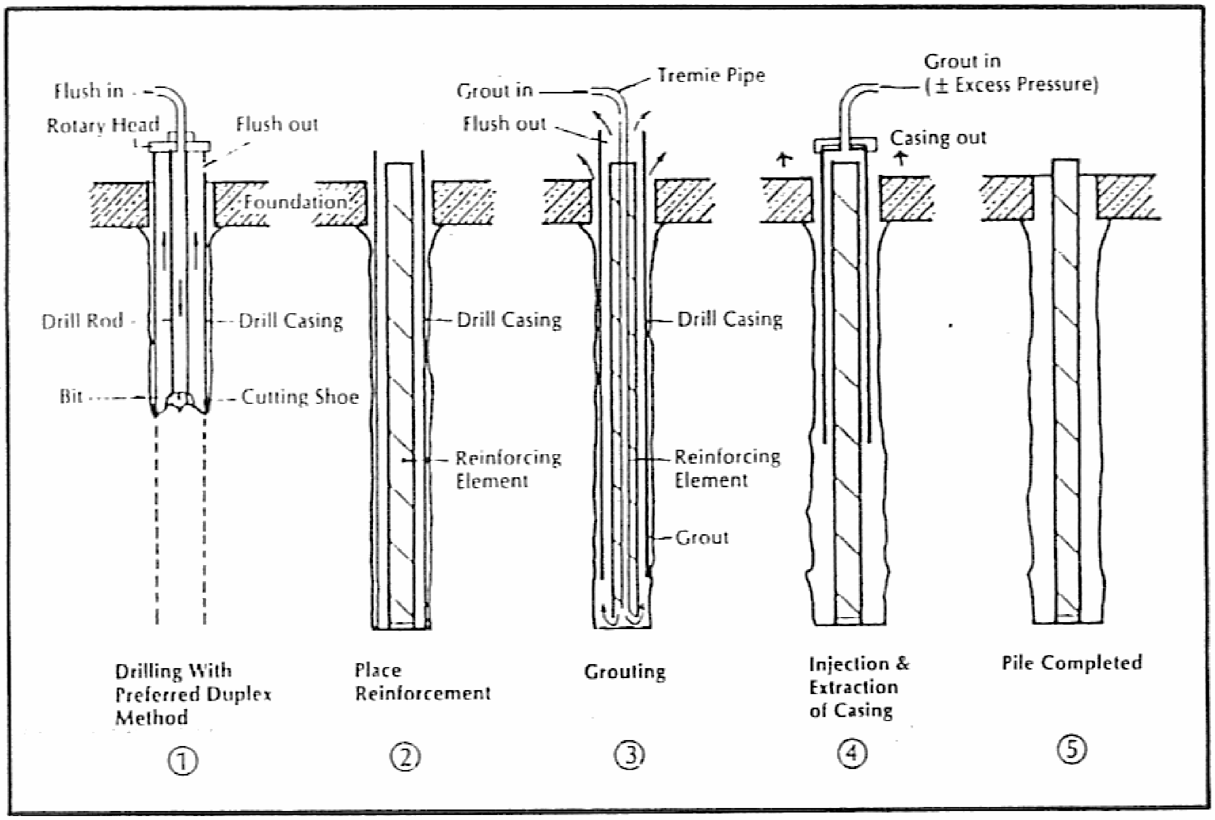


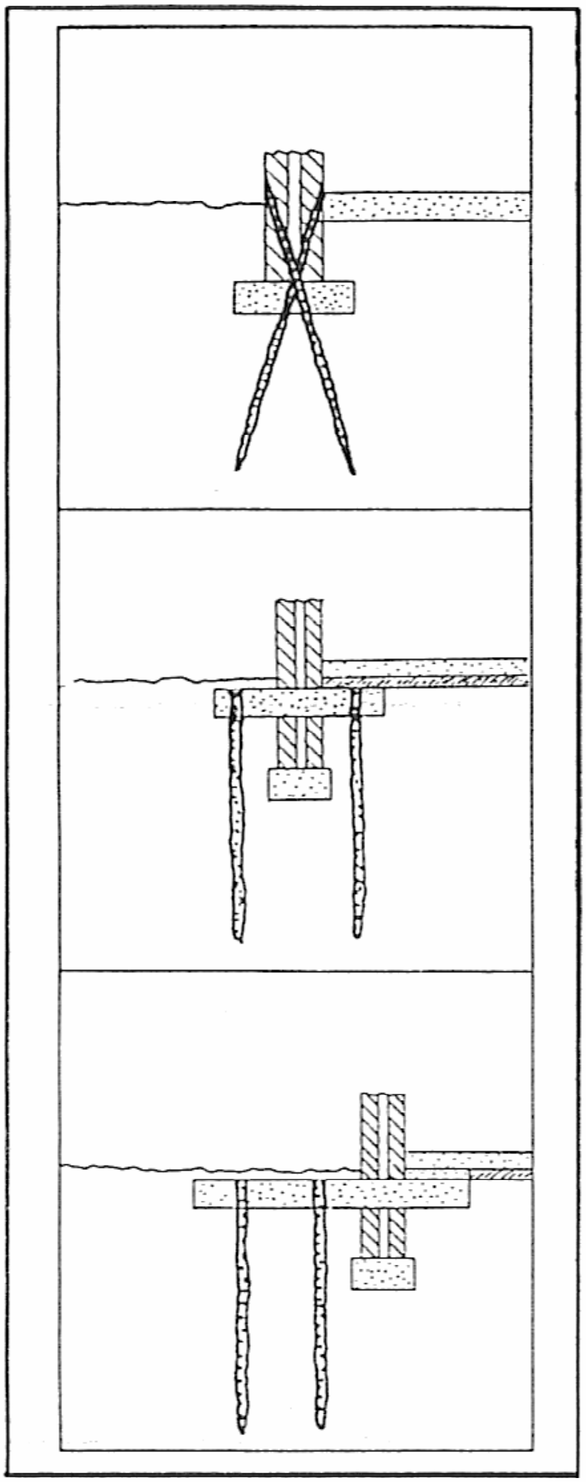
Figure 1 Stages in the construction of a standard pin pile (After Koreck, 1978)

- Connection to structure : adequate bond or connection must be provided in order to properly transfer loads (see Figure 2)). 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 owners 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 3. 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 2 cm is provided to the steel.

2.3. 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 displacement needed to mobilize frictional resistance are much smaller than those needed to develop end bearing.



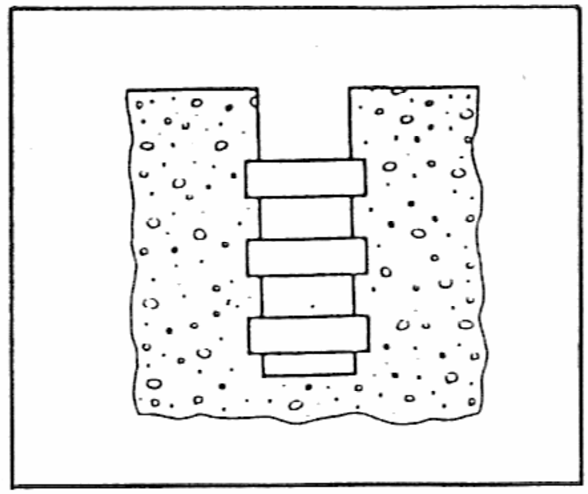


Figure 3 (Above) Typical triple-groove bonding profile produced by the Ankerbonder system in a diamond cored hole. (Anon, 1987)

Figure 2 (Left) Basic methods of load transfer from pin piles to structure.

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 4), 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.

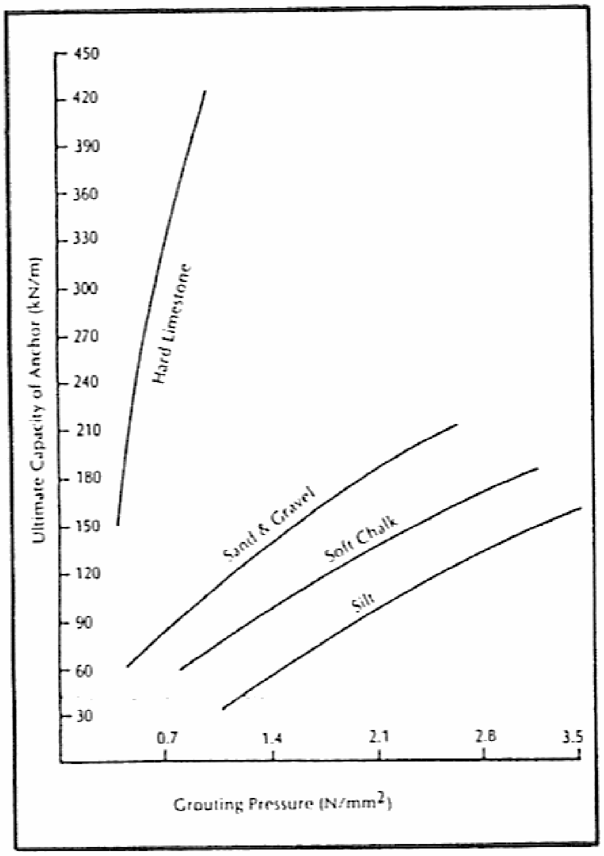


Figure 4 Influence of grouting pressure on ultimate load holding capacity. (Littlejohn and Bruce 1977)

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, therefore, always be made to local construction regulations for guidance, although the special aspects of pin piling 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 their stability 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 0.5 N/mm^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, 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 mini 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 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, Lizzi, and others, including ASCE, (1987) refer to the "knot effect" whereby a "positive" group effect is achieved in the loading of the soil-pile system. For example, Plumelle's (1987) full-scale testing yielded the results shown in Figure 5 that confirmed Lizzi's earlier model tests (see Figure 6). 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 on when preloaded piles are used.

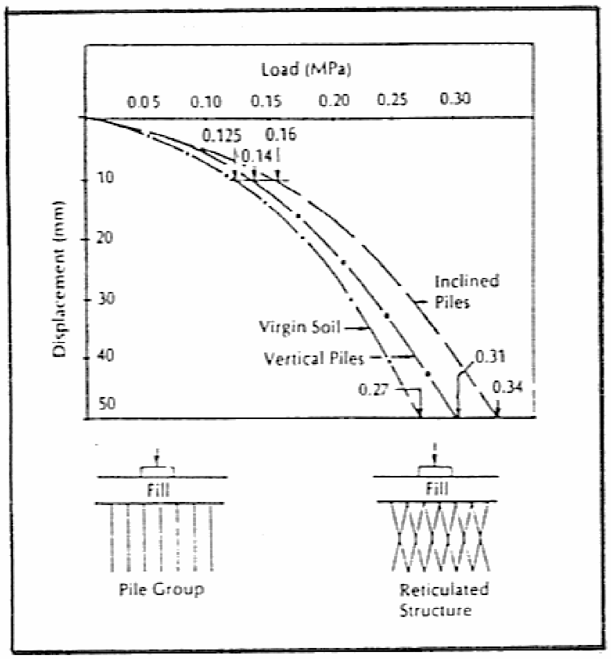


Figure 5 Field test data for different pin pile arrangements (Plumelle, 1984)

2.4. 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 groups merit special attention:

1. Post-grouting of bond zone
2. Reinforcement of free zone
3. 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

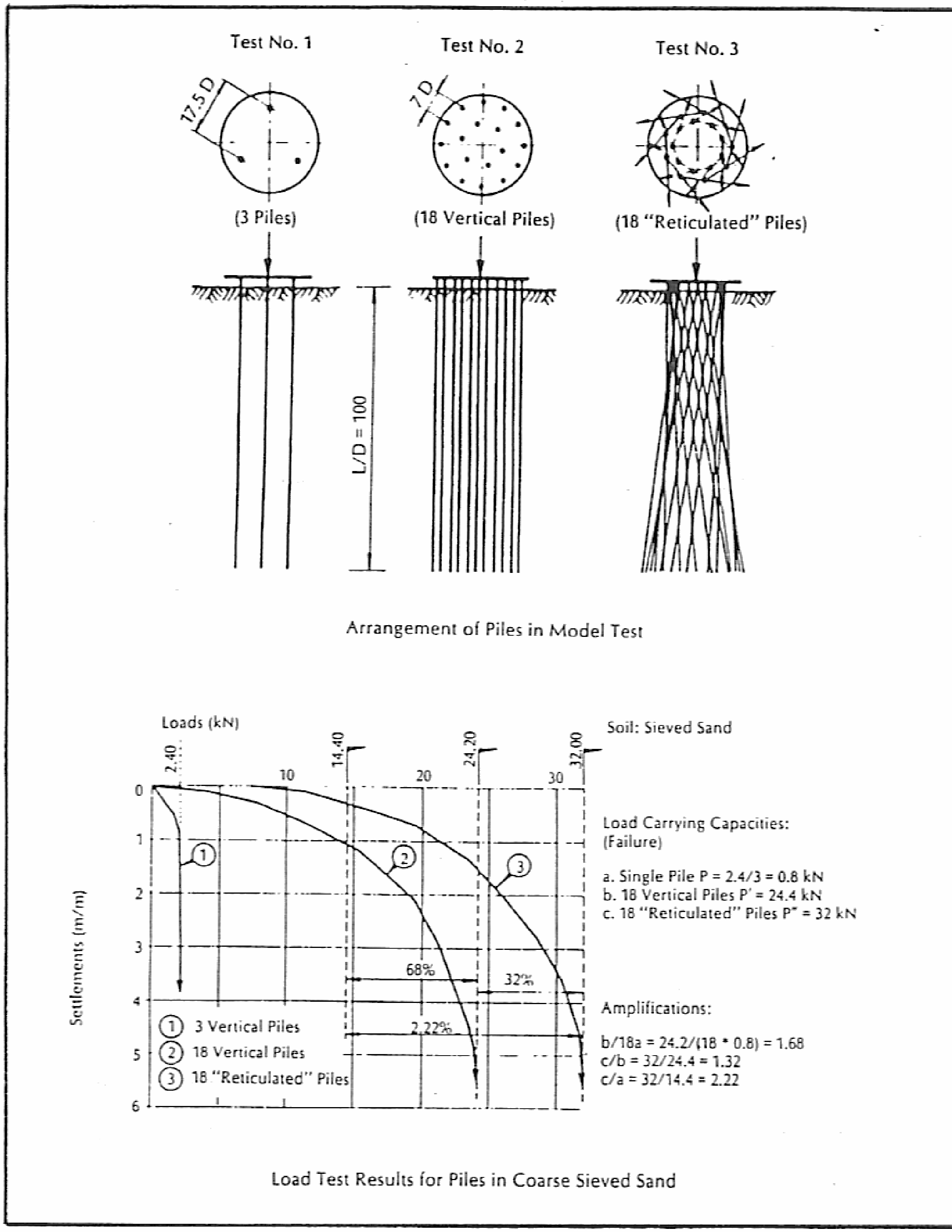


Figure 6 Model test data for different pin pile arrangements in coarse sieved sand (Lizzi, 1978).

(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 rubber sleeved ports at regular intervals (see Figure 7). 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. Pressures of up to 2 N/mm^2 are commonly used.

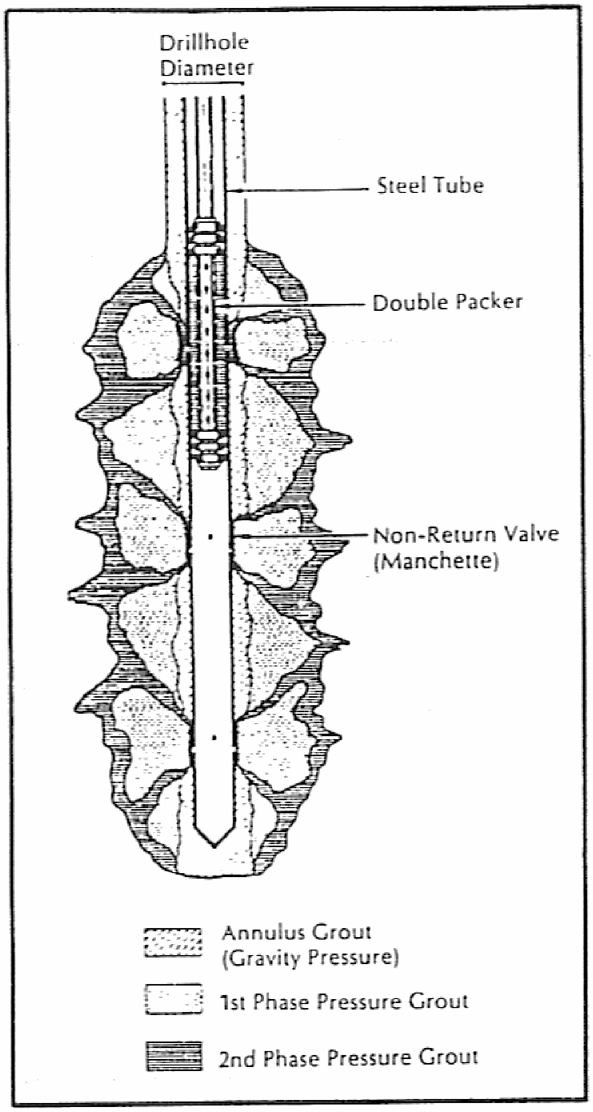


Figure 7 Concept of repeated post-grouting in increasing effective pile diameter (Mascardi, 1982).

Mascardi (1982) noted that in cases of repeated post-grouting, an effective pile diameter in the range 30 to 80 cm may be expected, considering that standard mini 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 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. In this case, the drill casing may be pushed back down into the bond zone (3 to 6 m) 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 pile structural failure, and prevents the wasteful travel of grout into often very permeable upper horizons. The load/settlement performance is also exceptional, as illustrated in Figure 8, 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 cm 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, using 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.

2.5. 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 (Bruce, 1988). For very heavily loaded structures (e.g., bridge piers), groups of pin piles are usually required. Even here, however, it would seem that the resulting group effect is positive, as opposed to demanding reduced individual 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.

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.

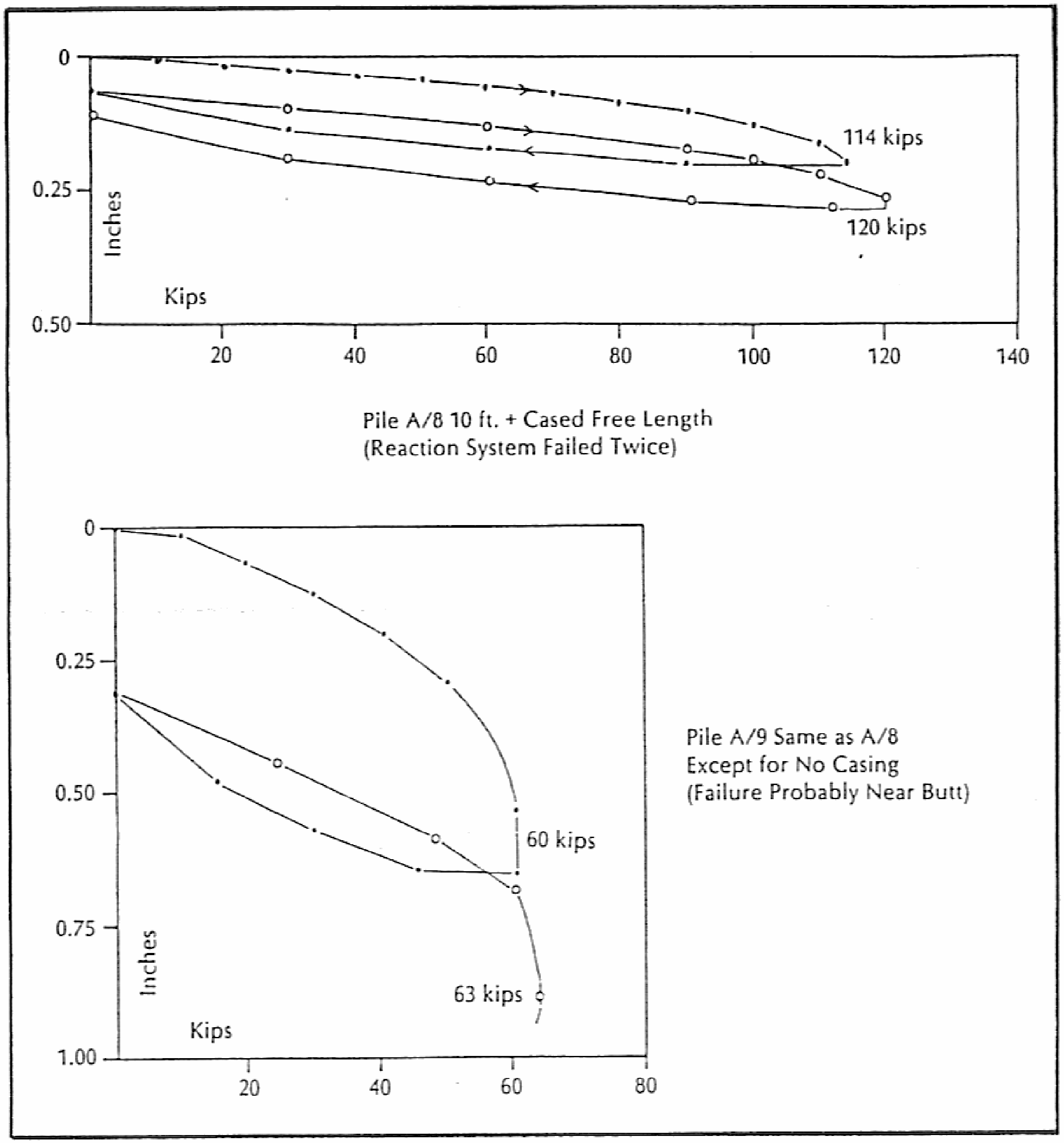


Figure 8 Pin pile load test data from a site in Coney Island, NY, showing the benefits of a steel liner through upper strata above the bond zone.

2.6. Case History

The existing derelict six- and three-story commercial buildings at 739 and 749 Boylston Street, respectively, in the Back Bay area of Boston were scheduled for redevelopment and refurbishing. The structures dated from 1902 and were founded on pile caps bearing on driven wooden piles. To accommodate the increased loadings resulting from the new construction, additional piles were required under enlarged pile caps. The owner accepted an alternate bid involving the installation of 260 pin piles with a working load 40 tons in compression and 12 tons in tension. The access, headroom and environmental restraints ruled out the use of driven wooden piles, while the overall reconstruction program demanded a short piling phase.

The piles served to transfer load through saturated fill (loose fine sand, trace silt), and organic deposits (soft silt with traces of shells, fine sand and fine gravel) into the bearing horizon - a medium dense to dense fine-coarse sand with traces of gravel and silt. The upper soft deposits were typically 3 to 4 m thick, while the sand, overlying the Boston Blue Clay varied from 6 to 8 m in thickness. In order not to penetrate into the clay, the maximum length of the piles was 8.4 m.

The piles were designed to have an ultimate load of 2.3 times the working load (i.e., 92 tons in compression and 27 tons in tension). Two test piles were installed and tested to verify the design assumptions, especially the adequacy of the 4.6 m bond length of anticipated grouted diameter 18 to 20 cm. Each pile was equipped with a steel casing from the surface to within 2.3 m of its tip to ensure proper performance even in the very soft upper materials, and a full-length centralized steel rebar. Pressure grouting up to 0.7 N/mm^2 with neat cement grout was conducted during the withdrawal of the temporary drill casing in the sand horizon. A telltale anchored near the tip of the pile was also installed in each test pile, which was also debonded from the upper horizons by an outer steel liner.

Reaction for each test was provided by adjacent ground anchors, and the tests were executed in accordance with the recently proposed modifications to the Massachusetts State Building Code (19894) and ASTM D1143 (1981). The data are summarized in Table 1, while the performance of test pile no. 2 (in compression) is shown in Figure 9 together with that of a timber pile for comparison.

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 the recommended state building code criteria for maximum net settlements. The performance in tension was equally satisfactory.

Production piles were therefore installed in the same way. Particular care was taken to ensure that the pressure grouted zone was correctly located in the sand. Timber and other obstructions were recorded in a limited number of holes, and were accommodated by the drilling method or (in three cases) by relocation. Other potential problems arising from access, and difficult site conditions were also easily dealt with. In total, 262 piles were installed in about 5 working weeks, substantially ahead of the anticipated program.

	Butt (cm)		Tip (cm)	
	Test Pile 1	Test Pile 2	Test Pile 1	Test Pile 2
Compression Test				
Gross Settlement	1.120	0.861	0.800	0.478
Net Settlement (Permanent)	0.640	0.394	0.638	0.411
Tension Test				
Gross Heave	0.605	0.358	0.442	0.165
Net Heave (Permanent)	0.419	0.221	0.391	0.165

Table 1 Test pile performance data. Boylston Street, Boston, MA.

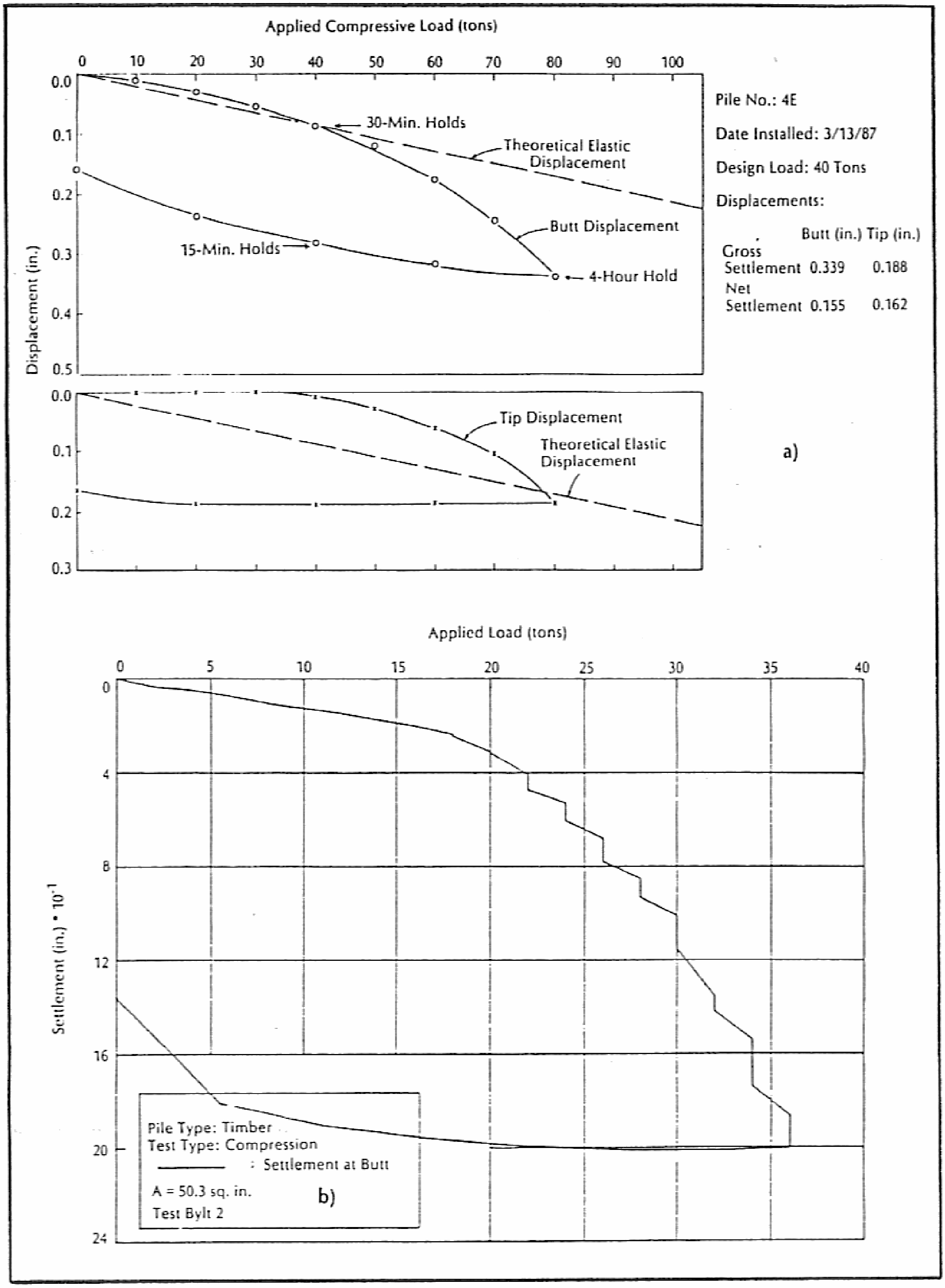


Figure 9 Load-settlement performance of a) a drilled and grouted pin pile, and b) a driven timber pile, Boylston Street, Boston, MA.

3. INSITU REINFORCING

There are three basic classes of in situ reinforcement (see Figure 10):

Soil nailing refers to reinforcing elements installed horizontally or sub-horizontally into the cut face, as top down staged excavation proceeds. The inserts improve the shearing resistance of the soil by being forced to act in tension.

Reticulated micropiles are similar inserts, but steeply inclined in the soil at various angles, both perpendicular and parallel to the wall face. The overall aim is to provide a stable block of reinforced soil to act as a gravity retaining structure.

Soil dowelling is applied to reduce or halt downslope movements on well defined shear surfaces. The principle exploits the large lateral surface bearing area and high bending stiffness of the dowels that are of far larger diameter than nails or pin piles (seldom greater than 15 cm). The use of soil dowelling is rare in urban environments, although it can prove attractive when combined with linked deep drainage in arresting massive land movements (e.g., in eastern Italy and southern California). (Bruce and Boley, 1987)

In terms of construction, soil nails and reticulated micropiles generally reflect the procedures of pin piles. Soil nailing is currently the more popular option in urban environments, but their geometry alone is so distinctive that there can be no confusion with load bearing piles. Reticulated inserts have a reputation in rural settings but are enjoying increasing success in difficult urban construction problems. They are the subject of this section.

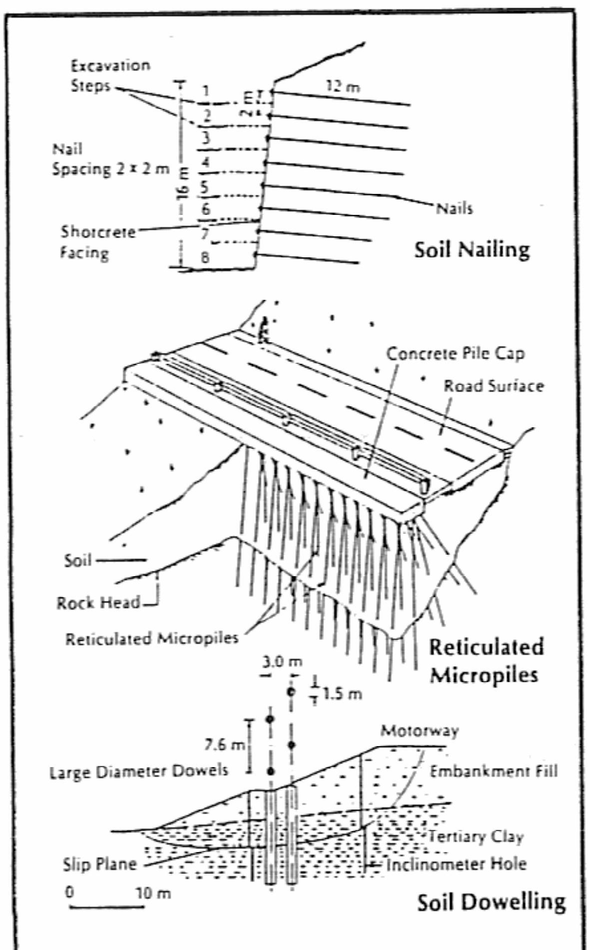


Figure 10 The family of in situ soil reinforcement techniques. (Bruce and Jewell, 1986)

3.1. Evolution and Applications

The technique of micropiling was first conceived, and patented, in Italy in 1952. The early applications were as direct underpinning, and often involved closely interlocking arrays of piles in which a positive "group effect" was achieved. Later, the system was applied as in situ reinforcing to solve slope stability problems, typically in rural areas. Such examples in the U. S. are listed in Table 2 while Lizzi (1982) provides numerous case histories of successful applications throughout Western Europe.

With particular reference to urban engineering, the major applications are illustrated in Figure 11 for cut and cover as well as bored tunnel construction. In each case, the concept is to create protective structures in the ground in order to separate the foundation soil of the building from the zones that are potentially subject to disturbance. Attwood (1987) describes the application of this system in the widening of a major roadway near London, where the very restricted access at the top of the embankment (1.75 m from cutting to existing structure) ruled out "conventional" techniques (Figure 12a). Where the structure had to act as a retaining wall for a bridge abutment (Figure 12b),

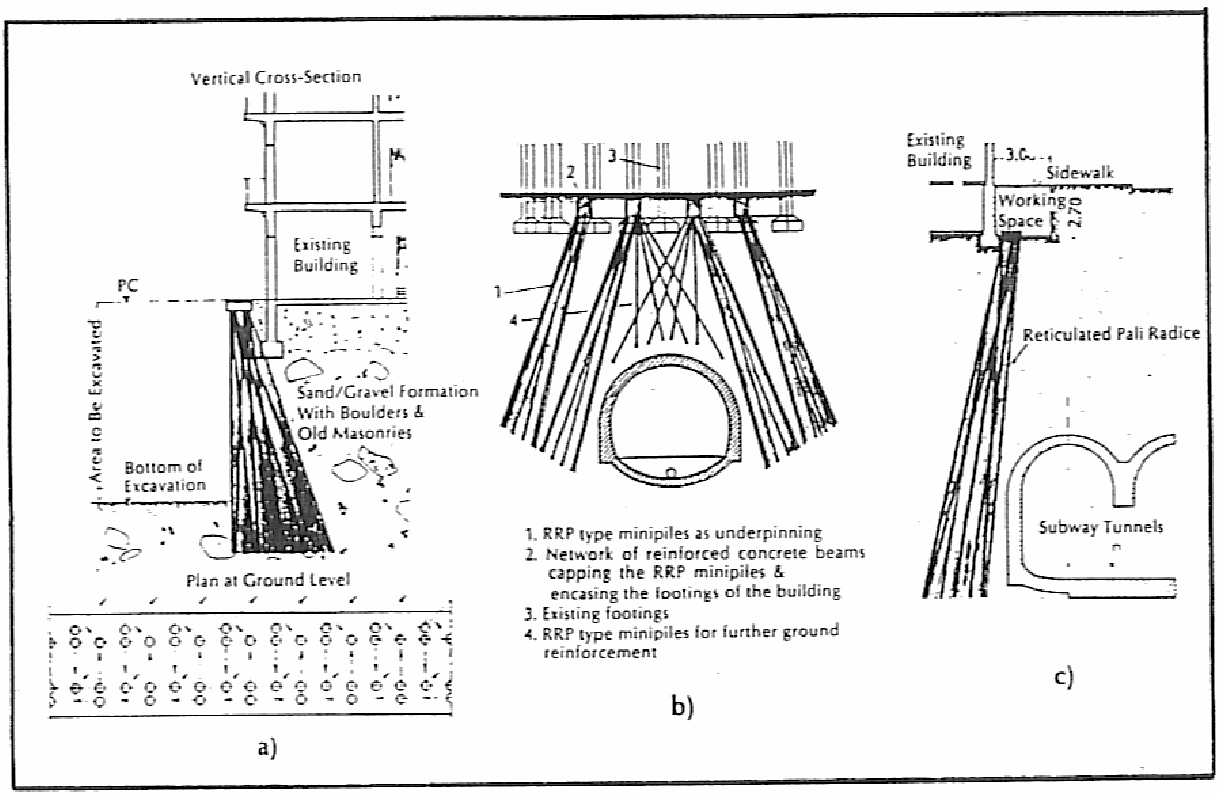
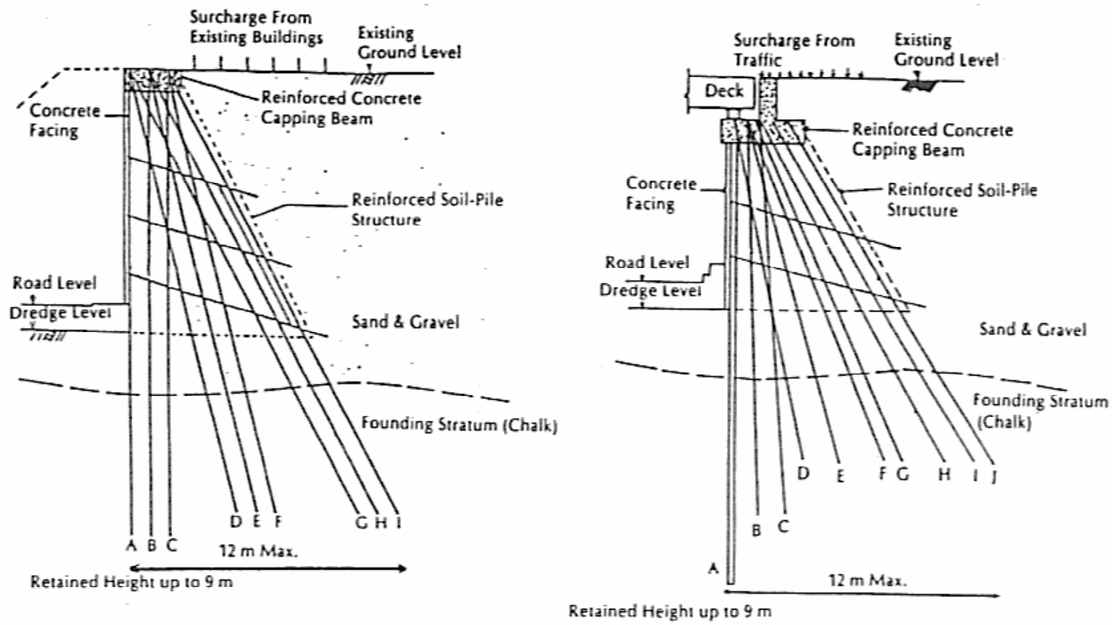


Figure 11 Applications of reticulated micropiles used as in situ reinforcement a) for cut and cover excavation, and b) around bored tunnels. (After Lizzi, 1982)

higher capacity tubular mini piles (40 tons WL) were incorporated into the vertical front face. Inclinometer readings during and after construction showed a maximum outward deflection of 1.5 cm for the 8 m high cut, with a profile resembling a typical anticipated cantilever performance.

						Pile Data								
Project Name & Location	Construction Date	Ground Conditions	Upslope/ Downslope Grade (U/D)	Depth to Slip-Surface at Wall (ft)	Cap Beam Geometry LxWxH (ft)	Diameter (in.)	Reinforcement Type & Grade	Average Number of Piles per Foot Length of Wall	Orientation (Degrees from Vertical) Upslope	Maximum Length (ft)	Depth Below Slip-Surface (ft)	Effective Wall Width along Slip-Surface (ft)	Measured Lateral Movement (in.)	Comments
Forest Highway 7, Mendocino National Forest California	1977	Micarous phillite $\phi = 13°$, c = 500 psf & schistose bedrock	2H:1V/2H	55	310x6x2.5	5	#9 Rebar Grade 60	2.25	19 0 -19	69 63 69	8	41.5	Not measured	Strain gage data indicate mostly compressive strains except near the pile cap
Route 23A Catskill State Pk., New York	1977	Moist very dense glacial till with boulders $\phi = 30°$, c = 0 & green & red shale	2H:1V/ 1H:1.3V	10-26	250x11x1.75	4 (min.)	#10 Rebar Grade 60	2.8	15 -15	40 50	30 24	33	Before cap 4 After cap 0.3	Designed as a space frame, the cap beam was constructed after the piles were finished
PA-306 Monessen, Pennsylvania	1979	Random fill colluvium & weak red shales $\phi = 17°$, c = 100 psf γ = 134 pcf	2.5H:1V/ 4H:1V	20	200x6x2.5	5	#9 Rebar Grade 60	2.25	19 -19	45	9	20	After completion 0.1 - 0.75	Slope downhill of wall failed after completion, no significant movement of piles was noticed
L.R. 69 Armstrong County, Pennsylvania	1985	Random fill underlain by wet sandy clay & a hard clay residual of bedrock $\phi = 30°$, c = 0 γ = 120 pcf	Level/ 1.2H:1V	34	310x6x2.5	6	#11 & #14 Rebar Grade 60	1.33	40 -8	50	15	20	After completion 0.125	Piles oriented at 40° back from vertical were designed as tiebacks to satisfy rapid drawdown condition, section south of insert wall failed in 1987 but without effect on stabilized section

Table 2 Summary of reticulated micropile wall case histories (U. S. Applications). (Bruce, 1988)



Typical Range of Pile Loadings (kN)

a)

	Max	*Min*
Row A	165	90
B	153	65
C	142	40
D	134	23
E	118	-10
F	103	-43
G	87	-77
H	72	-110
I	56	-144

b)

Row A	366	311
B	125	57
C	114	33
D	105	17
E	99	0
F	84	-33
G	77	-49
H	62	-81
I	51	-106
J	40	-130

Figure 12 In situ reinforcement for road widening project, Dartford, London a) normal retaining wall, and b) retaining wall serving as bridge abutment (Attwood, 1987)

3.2. Construction

The drilling and grouting equipment and systems are generally the same as those used for soil nail and ground anchor construction. Unlike nails, however, pressure grouting with neat cement or sand-cement grouts is usually conducted to pressures of up to 1.2 N/mm^2. The concrete capping beam merits particular attention during design and construction, although it can be completely buried after construction. There are indications that if the beam is cast before drilling is commenced, there is less movement of the reinforced soil mass during and after construction.

3.3. Design and Performance

The basic design concept is that the structure acts as a gravity wall, consisting of a monolithic mass. This mass relies for its continuity on soil-insert interaction and not on intergranular cementation. The soil itself supplies the gravity, while the inserts supply the "lines of force" that allow the whole mass to support compression, tension and shear (Lizzi 1982). The completed wall should thus physically prevent loss of soil from behind, and prevent sliding along potential failure planes.

Instrumented field programs have confirmed that reinforcement stresses and overall wall movements are minimal, and that most probably the designs to date have been highly conservative. This conservatism clearly reflects the current lack of a rigorous design approach. Lizzi (1982) confirmed "it is not yet possible to have at our disposal an exhaustive means of calculation ready to be applied with safety and completeness." Lizzi noted that his "paper does not suggest formulas or elaborate mathematical calculations because it cannot offer theoretical approaches in the usual way geotechnical problems are dealt with." In addition, the ASCE Committee (1987) also alluded to the great reliance placed in designs on the soil/pile interaction "which is still subject to experience and intuition."

In brief, the design approach is to calculate the contribution offered by the piles to the resistance of the natural soil. The factor of safety, F', is calculated as:

$$F' = (R + R')/A$$

where:

F' = Factor of safety
R = Soil shear resisting forces on the critical surface
R' = Additional shear resistance supplied by the piles
A = Total driving forces along the critical surface

R' is the allowable stress in the grout and reinforcement in the pile cross-section, or the shear resistance of the soil along a section of the pile mobilized by the sliding mass (whichever is less), multiplied by the number of piles on the critical surface. This approach is considered conservative, since it does not take into account the interaction developed between the soil and the pile.

As an example, the general design for the Mendocino wall involved:

- Estimating loads (active and passive) on the wall.

- Conducting a stability analysis to determine the shear force needed to maintain a required factor of safety.

- Determining the number of piles needed to provide the required shear resistance.

- Calculations (similar to those for a conventional gravity wall) to check stability against overturning, sliding and bearing failure at the base of the wall.

The recorded design procedure included extending the piles into rock where possible (and always below the failure plane). However, a valid comment on design was made by Dash (1987), who reaffirmed that no checking is done to ensure that the reinforced mass really behaves like a composite material. He conceded that it was "very difficult" and "controversial," contrasting the density of 1.33 piles per foot (L. R. 69 in Table 2) with 2.25 at Monessen, Pennsylvania, used by different contractors. Based on "our present experience," he proposed a chart to aid in the selection of pile spacing and diameter (see Figure 13).

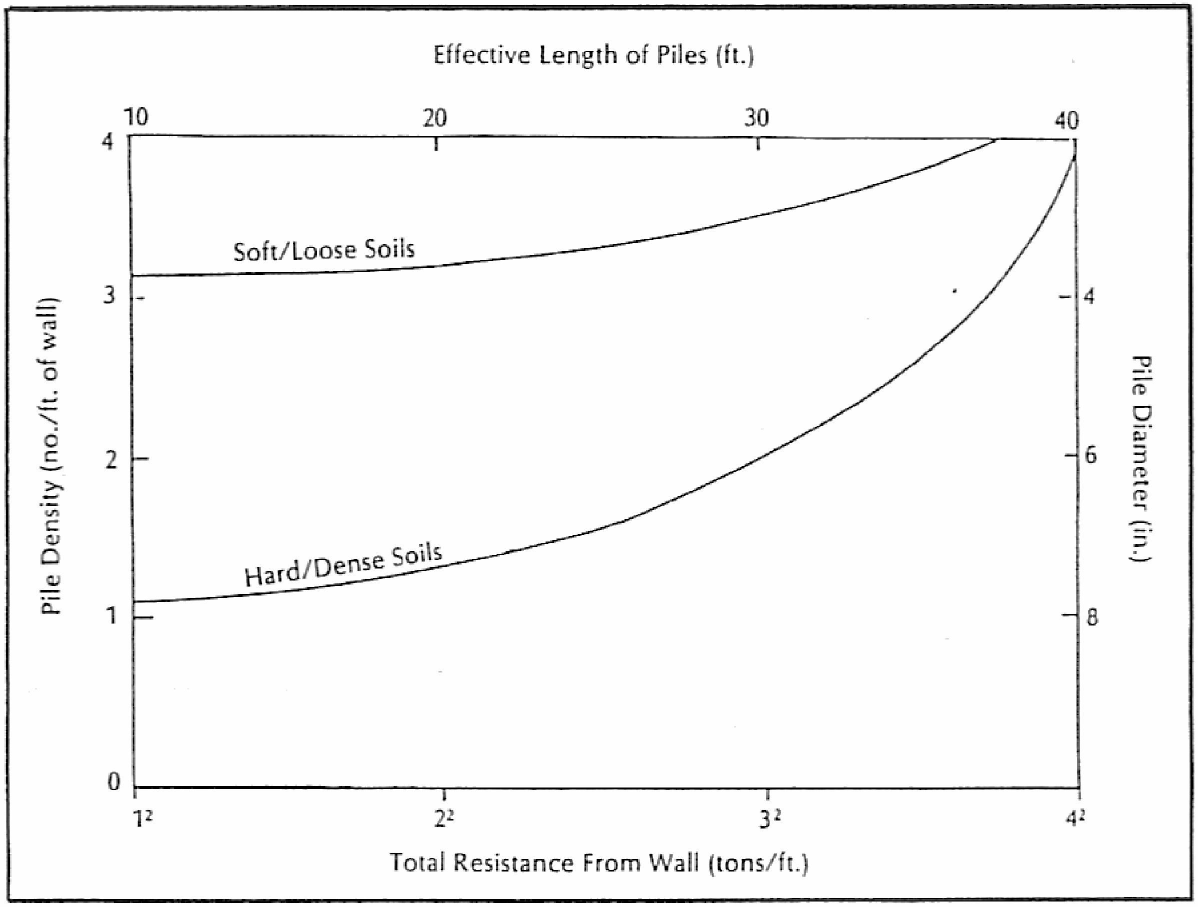


Figure 13 Chart for the preliminary design of reticulated micropiles (Dash, 1987)

3.4. Benefits and Limitations

In situ retaining structures, as shown in Figures 11 and 12 can be constructed in close proximity to existing buildings and in relatively tight access locations without the need to excavate and without causing any decompression of the foundation soil. Given their mode of construction, they can be installed in any type of ground, including boulders, old foundations or any other obstructions with no constructional limit placed on hole inclination or orientation.

There are two major restrictions that have limited the popularity of in situ retaining structures in the U. S. so far. First, as described above, there is no rigorous design methodology; and second, the intensity of the inserts installed has often impaired their cost effectiveness. However, fundamental research is underway to try to understand better the soil-structure phenomena. This research should lead logically to a more precise design approach, and to more economic solutions resulting from a reduction in the intensity of the reinforcement deemed necessary. These two consequences should result in a greater acceptability of the technique in the U. S.

3.5. Case History

Legislative Route 69 between Vandergrift and Leechburg, PA runs along the Kiskiminetas River. Much of it is cut into the steep hillside along the bank of the river, with the cut material used as downhill side fill to provide the highway bench. In a number of places this embankment material proved unstable and slides occurred. One such slide repair was designed by The Pennsylvania

Department of Transportation (PennDOT) as a tie-back caisson wall. This project was bid in the Summer of 1984 with bids accepted on the "as designed" wall or contractor designed alternatives. The author's company was successful based on an alternate design (Figure 10b). The 95m long slide had previously been repaired by filling and repatching, as the closeness of the river prohibited placing a fill buttress below. Given its role as a major traffic route, detours would have proved lengthy and inconvenient and so the stabilization method had to allow passage of one lane of traffic throughout.

The design height for both the as-designed wall and the insert wall, was 10m. The spacing on the inserts (Figure 14) was chosen to achieve the maximum width at the top of rock for stability purposes. It was determined that the minimum density would be one reinforcing unit per 2 square meters at the top of rock. The resultant width of the gravity mass at the top of rock was approximately 6m. The stabilized mass thus achieved was, however, still potentially unstable with respect to overturning during maximum rapid drawdown conditions following a flood

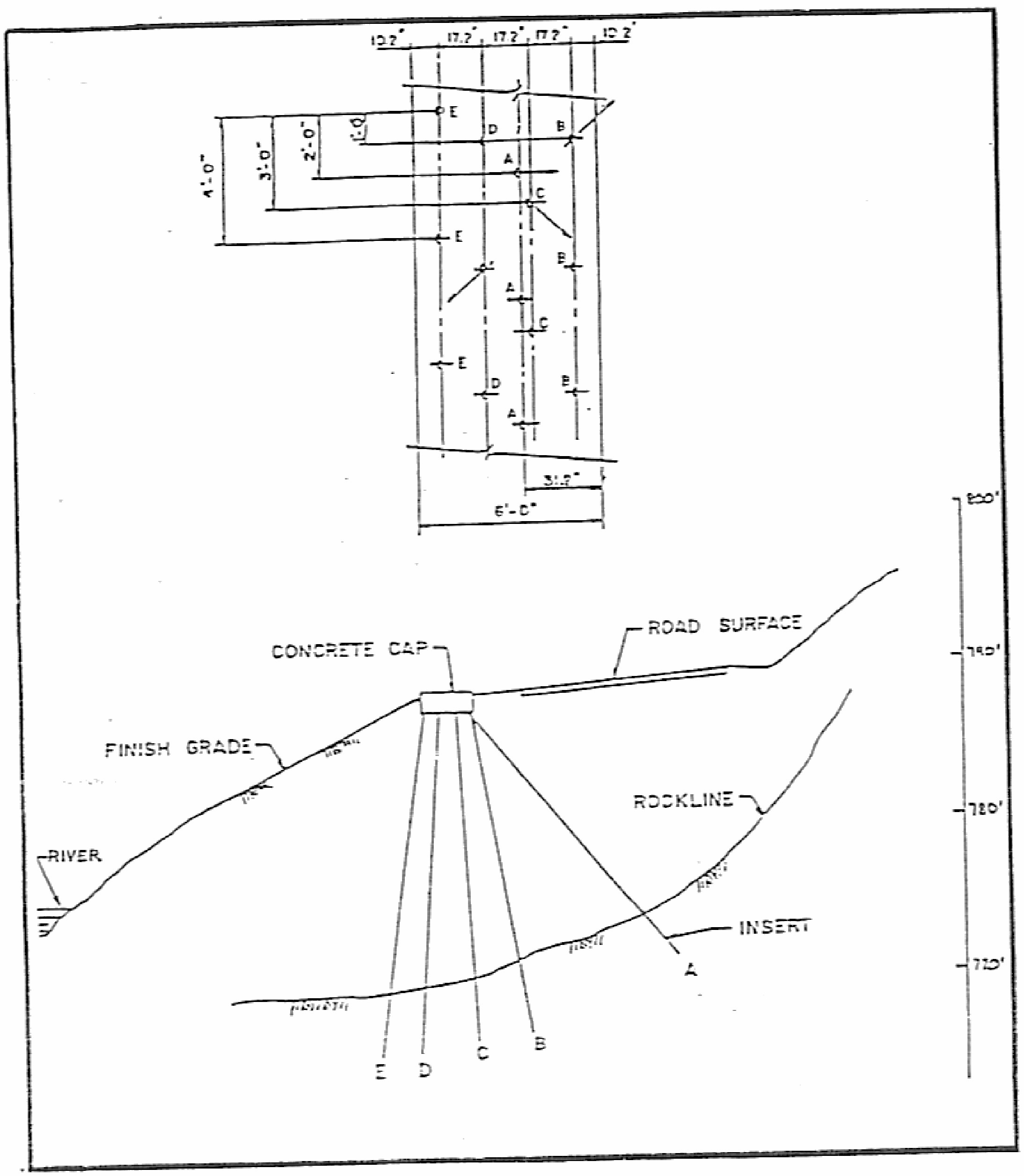


Figure 14 Details of in situ reinforcement at LR 69, PA. (Boley and Crayne, 1985)

For this reason, an additional row of tension members was added (Row A, Figure 14), of full design load 40 tons. These additional tension members gave a minimum factor of safety of 1.24 during rapid drawdown conditions and a factor of safety against overturning during steady-state seepage conditions of 1.95. A density of 1.33 nr 15cm diameter holes per lineal foot of wall satisfied all the stability criteria, when equipped with #11 or #14 rebar, grouted with a neat cement mix to $0.3N/mm^2$ excess pressure.

All the inserts were extended into rock a minimum of 4.5m to ensure that they would function as shear keys to thus prohibit sliding of the gravity mass and also to allow the transfer of any axial load, whether in tension or compression, into bedrock. Special diesel hydraulic drilling rigs were used to ensure fast penetration through both soil and rock in one pass.

Monitoring of the slide by slope indicators during 1984 showed movement of the slide at a depth of 9m of about 1.2cm during a seven-month period. The slope indicator readings taken between August and November at 9m depth showed additional movement of about 9cm. On about November 10, 1984, after the concrete cap had been placed and the drilling of the inserts has just started, some additional movement occurred. A section of roadway along almost the entire length of the cap moved vertically downward about 10cm and a crack about 2cm wide opened up in the middle of the road surface. No movement or cracking of the cap was detected so installation of the inserts in the slide area was accelerated to provide support as soon as possible. About a week later, after installation of about 40 of the inserts, movement appeared to have stopped. No further movements of the roadway have been detected and other slope indicator readings taken to date have shown little or no further movement.

4. CONCLUDING REMARKS

Pin piles and inserts used for insitu reinforcement share many physical and construction characteristics. However, their purpose, design, and performance are fundamentally different, and this has not often been clearly stated in previous publications. Each technique has great potential in the field of urban engineering, offering a great deal of flexibility in problem solving to both designer and constructor alike. It is hoped that this paper will help to rationalize thinking on the use of small diameter cast-in-place inserts, and so contribute to the continuing expansion of their markets.

REFERENCES

Pin Piles

Anon, "Core Drilling Through Concrete Pier Bases Preparatory to Mini Piling," Geodrilling, August 1987, pp. 16-18.

ASCE Committee on Placement and Improvement of Soils, Soil Improvement - A Ten-year Update, Proceedings of Symposium at ASCE Convention, Atlantic City, New Jersey, April 28, 1987.

American Society for Testing and Materials, Testing of Piles Under Static Axial Compressive Load, D1143-81, 1981.

Bjerrum, L., "Norwegian Experiences with Steel Piles to Rock," Geotechnique, Vol. 2, No. 2, 1957, pp. 73-96.

Bruce, D. A., Ingle, J. L., and Jones, M. R. "Recent Examples of Underpinning using Mini Piles." 2nd Intl. Conf. on Structural Faults and Repairs, London, April, May 1985, 11 pp.

Bruce, D. A., "Aspects of Mini Piling Practice in the United States." To be published in "Ground Engineering." November 1988.

BSI, CP 2004 Foundations, London, 1972.

Commonwealth of Massachusetts, State Building Code. Fourth Edition, 1984.

Gouvenot, D (1975) "Essais de chargement et de plambement de pieux aiguilles," Annales de ITdB et de Travaux Publics. No. 334, December 1975.

Johnson, E. G., and Schoenwolf, D. A. (1987). "Foundation Considerations for the Expansion and Renovation of the Hynes Auditorium," Civil Engineering Practice, Vol. 2, No. 2, Fall 1987, pp. 35-62.

Jones, D. A. and Turner, M. J., "Post-grouted Micro Piles," Ground Engineering, Vol. 13, No. 6, 1980, pp. 47-53.

Koreck, H. W., "Small Diameter Bored Injection Piles." Ground Engineering, Vol. 11, No. 4, 1978, pp. 14-20.

Littlejohn, G. S. and Bruce, D. A., Rock Anchors - The State of the Art, Foundation Publications Ltd., Brentwood, Essex, England, 1977, 50 pp.

Lizzi, F., "Reticulated Root Piles - to Correct Landslides," ASCE Convention, Chicago, October 16-20, 1978, Preprint 3370, 25 pp.

Lizzi, F., "The Pali Radice (Root Piles)", Proc Inl. Symp. on Soil and Rock Improvement, AIT, Bangkok, November-December 1982, Paper D-1, 21 pp.

Lizzi, F., "The Static Restoration of Monuments," Sagep Publisher, 1982, 146 pp.

Mascardi, C. A., II comportamento dei micropali sottoposti a sforzo assiale, momento flettente e taglio. Verlag Leemann, Zurich 1970.

Mascardi, C. A., "Design Criteria and Performance of Micropiles," Proc. Int. Symp. on Soil and Rock Improvement, AIT, Bangkok, November 29 - December 3, 1982 Paper D-3, 18 pp.

Munfakh, G. A. and Soliman, N. N., "Back on Track at Coney." Civil Engineering, December 1987, pp. 58-60.

Plumelle, C., "Amelioration de la portante d' unsol par inclusions de groupe et reseaux de micropieux," Int. Symp. on In Situ Soft Rock Reinforcement, Paris, October 9-11, 1984, pp. 83-90.

P.T.I., Recommendations for Prestressed Rock and Soil ANchors, Phoenix 1986, 41 pp.

Weltman, A., "A Review of Micropile Types." Ground Engineering, Vol. 14, No. 3, 1981, pp. 43-49.

In Situ Reinforcing

ASCE Committee on Placement and Improvement of Soils, Soil Improvement - A Ten-year Update, Proceedings of Symposium at ASCE Convention, Atlantic City, New Jersey, April 28, 1987.

Attwood, S. "Pali Radice: Their Uses in Stabilizing Existing Retaining Walls and Creating Cast in Situ Retaining Structures." Ground Engineering, Vol. 20, No. 7, 1987, pp. 23-27.

Boley, D. L. and Crayne, L. M. "INSERT Wall Saves Roadway." Highway Builder, Fall, pp. 14-16.

Bruce, D. A. and Jewell, R. A., "Soil Nailing: Application and Practice." Ground Engineering Vol. 19, No. 8, pp. 10-15, and Vol. 20, No. 1, pp. 21-32.

Bruce, D. A. and Boley, D. L., "New Methods of Highway Stabilization." Proc. 38th Annual Highway Geology Symp., Pittsburgh, PA, May 1987, pp. 57-72.

Bruce, D. A., "Developments in Geotechnical Construction Processes for Urban Engineering." Civil Engineering Practice, Vol. 3, No. 1, Spring 1988, pp. 49-97.

Dash, U., "Pin Pile Wall Evaluation." Research Report RR 84-36, prepared for U.S. Department of Transportation, FHWA under Contract DTFH 71-85-501-PA-14, March 1987, 174 pp.

Lizzi, F., "The Static Restoration of Monuments," Sagep Publisher, 1982, 146 pp.