American developments in the use of small diameter inserts as piles and insitu reinforcement

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The use of small diameter cast-in-place bored piles is becoming increasingly popular in the United States. Such inserts are used as conventional load bearing piles, and are generally referred to as pin piles. The other major application is for the in situ reinforcement of soil slopes or excavations. The paper reviews the applications, construction, design and performance of inserts of each type.

1. PREAMBLE

In the last decade or so in the United States, 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. It is important 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 may even have been fostered by certain European specialist companies in the earlier days of importing the techniques, in their desire to maintain some type of proprietary mystique. It is the intention of this paper to review both major groupings, and to clarify the fundamental differences between them at a time when the market potential of each group continues to expand.

2. PILING

Such inclusions used as piles are referred to generically as mini piles, while in the United States the term pin piling is achieving national recognition. Pin piling is now well into its fourth decade of application in Europe, although the U.S. expansion dates from the mid 1970's. Comprehensive general reviews of design, construction and performance are readily available (e.g. Koreck 1978, Weltman 1981, Lizzi 1982, Bruce et al. 1985, and Bruce 1988), and so this review concentrates on current North American trends.

2.1. Evolution and Applications

With the expiration of the original Italian patents in 1972, the use of mini piles 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 movements 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 Faber Cancer Institute where a total of 165 piles with a 75-ton service 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 service 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 service 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. Summary details of this project, and others executed recently by the author's company are provided in Table 1.

2.2. Construction

The most common basic method of installing mini piles remains as shown in Figure 1. Traditional variants using compressed air to pressurize the grout, or a vibrated mandrel (displacement pile), are described by ASCE (1987), but are not used in the U.S. Likewise, the "expanded base" pile (Lizzi, 1982) and the Menard inflatable cylinder pile (Mascardi, 1982) are never seen nowadays.

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

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	
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	
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

(Continued)

FOUNDING STRATA	GEOMETRY Length (ft)	Dia (in)	COMPRESSIVE LOAD (tons) Working/Test	APPLICATION	MINIMUM HEADROOM (ft)	LOCATION	DISPLACEMENT (in) at Test Load Total	Permanent
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	

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

Although certain authors try to differentiate pile nomenclature in terms of diameter (Weltman, 1981), it appears that all such piles that can be installed with conventional drilling and grouting equipment are referred to as mini 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 with the contractor but certain factors remain common with European practice:

-Drilling: a method is chosen to ensure minimal disturbance or upheaval to the structure or soil. Typically, duplex methods are used, and there is increasing use of the Doublehead System (Bruce 1984) for deep piles in especially difficult soils.
-Grouting: relatively high strength neat cement or sand-cement grouts are used at injection pressures rarely above 0.8 to 1.0 Mpa. Continuous pressure, progressive cavity pumps are favoured over piston types.
-Reinforcement: may be reinforcing cages (compressive loads only), high strength bars (compression or tension) or pipes (to resist bending stresses and enhance corrosion resistance).
-Connection to structure: adequate bond or connection must always 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 towers in England (Anon, 1987) and is being promoted in the U.S. 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. In other applications, horizontal post-tensioning of the foundation beam has been undertaken, providing a "clamping" effect to guarantee structure/pile continuity. The use of high strength, low shrink grouts for the grouting of the upper pile section, within the structure, is also common.

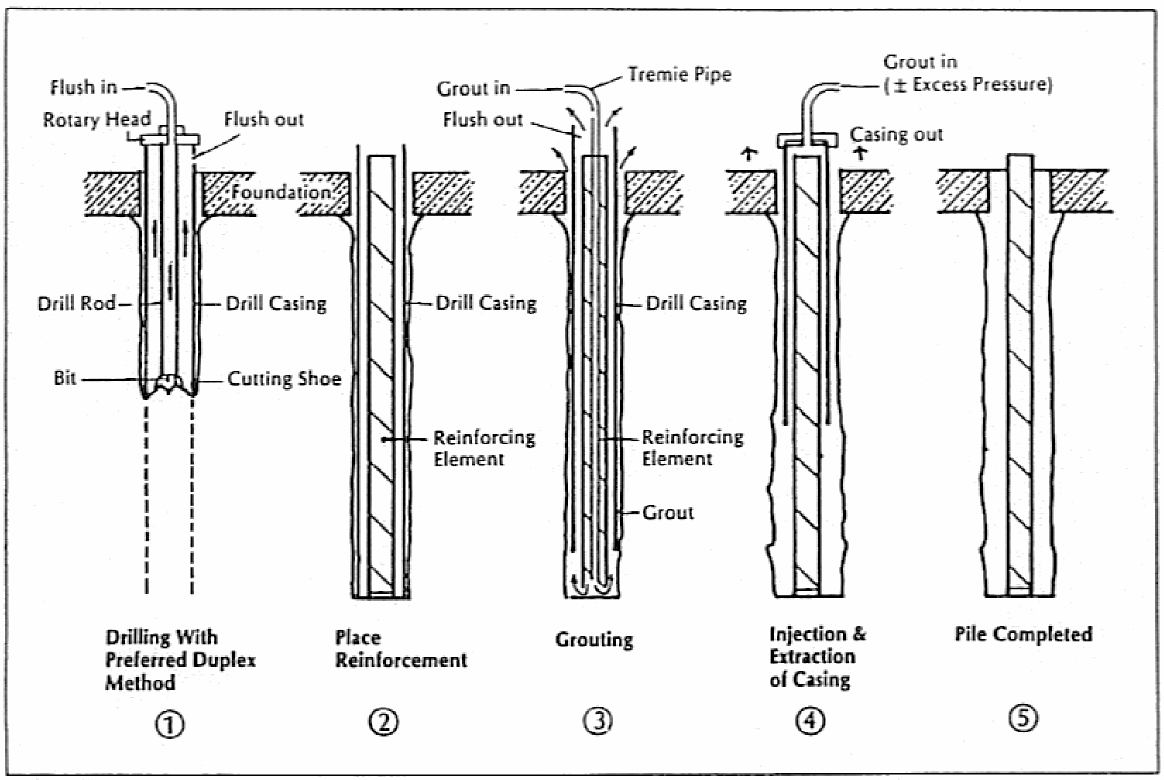


Figure 1 Stages in the construction of a standard pin pile (After Koreck, 1978)

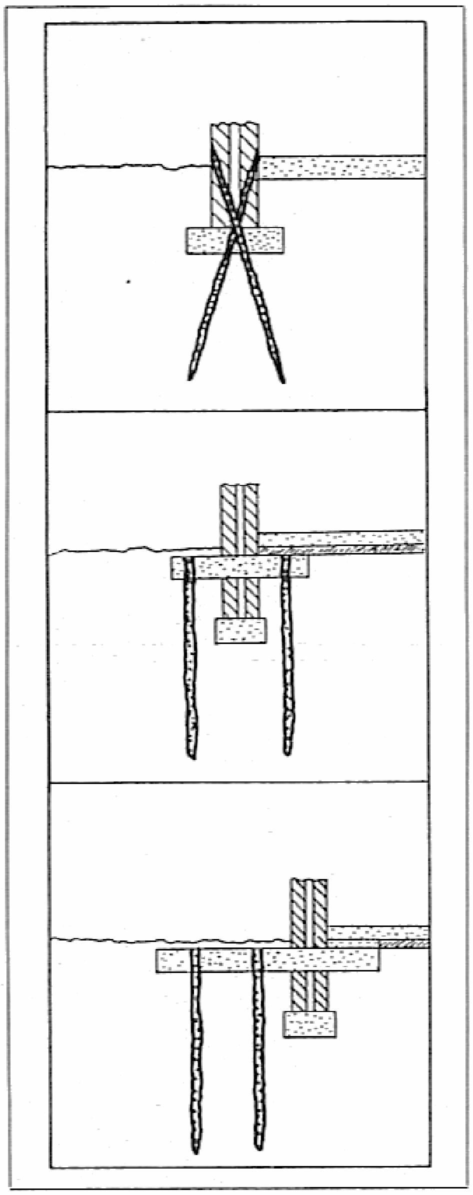


Figure 2 Basic methods of load transfer from pin piles to structure.

-Corrosion protection: when piles are required to act in tension, or when they are installed in particularly aggressive conditions, then particular attention is paid to the corrosion protection of the steel element. Similar to ground anchorages, protection in the form of an outer corrugated sheath is used, while centralizers are placed on the steel in the hole to ensure that a minimum grout cover of about 2 cm is provided to the reinforcement.

2.3. Design and Performance

Pin piles to be constructed wholly in soils 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 surprisingly good load/movement characteristics: the relative displacements needed to mobilize frictional resistance in soils are much smaller than those needed to develop end bearing (e.g. Figure 4). 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 are drawn with soil anchor practice (Figure 5), albeit for interfaces in the opposite sense of shear. As a general guide to the design of the transfer length, the Recommendations of the Post Tensioning Institute (1986), are typically followed.

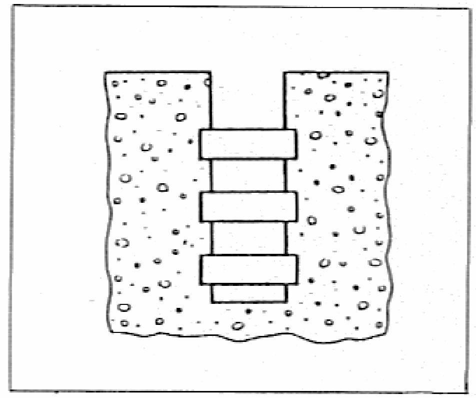


Figure 3 Typical triple-groove bonding profile produced by the Ankerbonder system in a diamond cored hole. (Anon, 1987)

When rock head is within reach, technically and economically, many American designs have featured piles bearing on, or socketted a short distance into, fresh rock. In such cases the piles act as simple struts, transferring compressive loads directly to bedrock.

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 movement limits, and in such a fashion that the elements of that system are operating at safe stress levels.

In certain cases, 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 of course be economically viable, and clients typically still acquire the reassurance of being able to review details from similar projects successfully conducted in the States when bids are being assessed.

Reference is always made to local construction regulations for guidance on design although the special aspects of pin piling are not generally adequately or specifically addressed, outside of Massachusetts (1984). In most cases, "sensible interpretation" is necessary.

Usually, 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 minipile design is most often limited by the internal carrying capacity. Emphasis is therefore placed on the steel and grout strength parameters as well as the grout/steel bond.

A final point may be made in relation to the "group effect" - a potentially very beneficial consideration in mini pile design but as yet infrequently exploited. The contrast with conventional piling is marked. For example, the

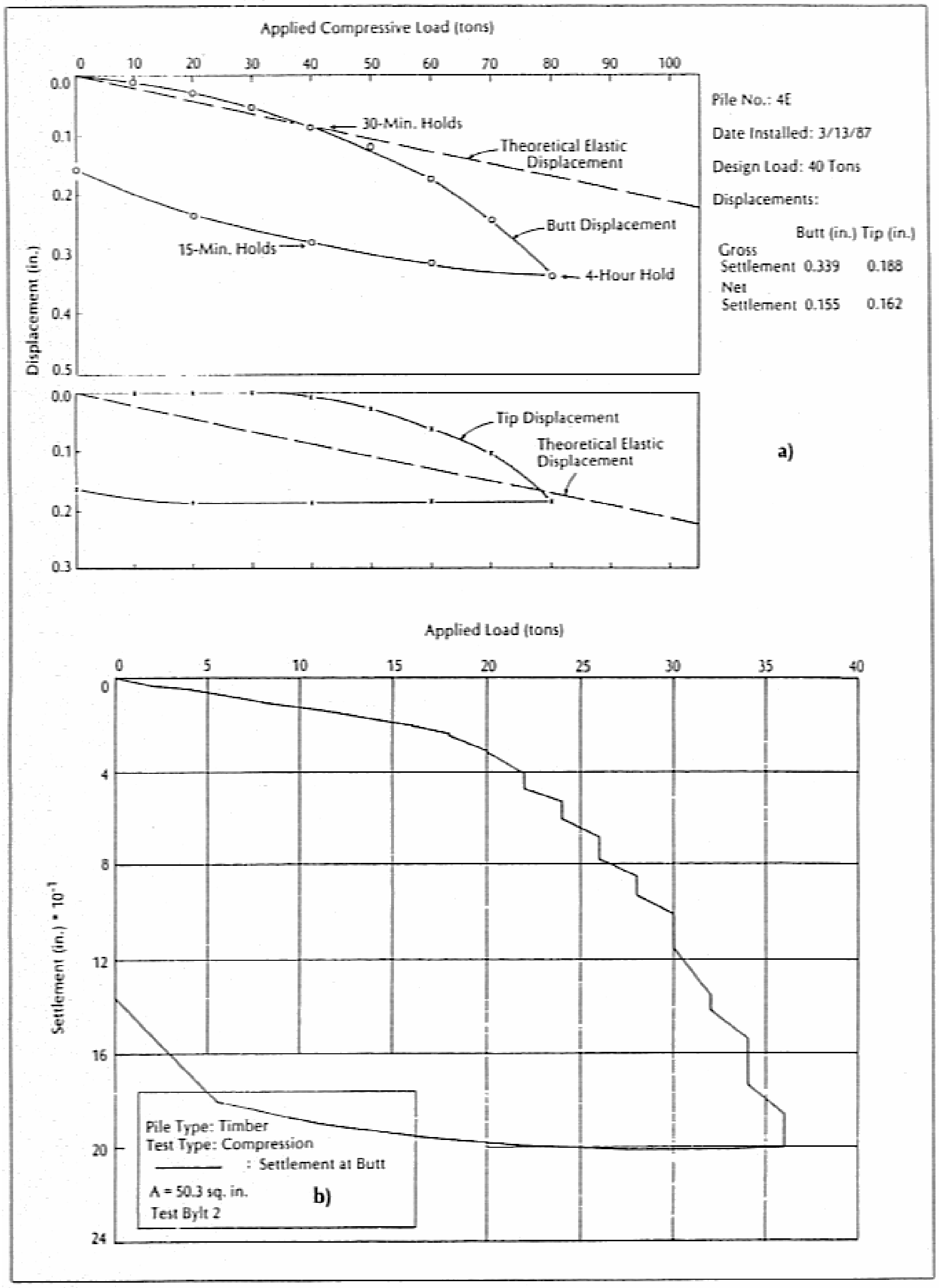


Figure 4 Load-settlement performance of: a) a drilled and grouted pin pile; b) a driven timber pile; Boylston Street, Boston, MA.

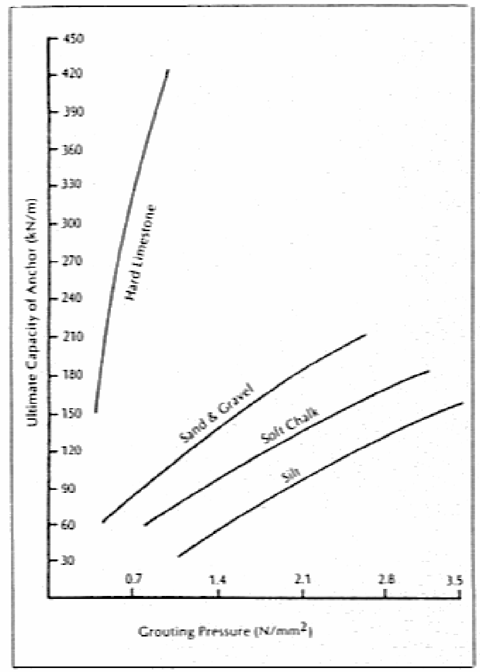


Figure 5 Influence of grouting pressure on ultimate load holding capacity.(Littlejohn and Bruce,1977)

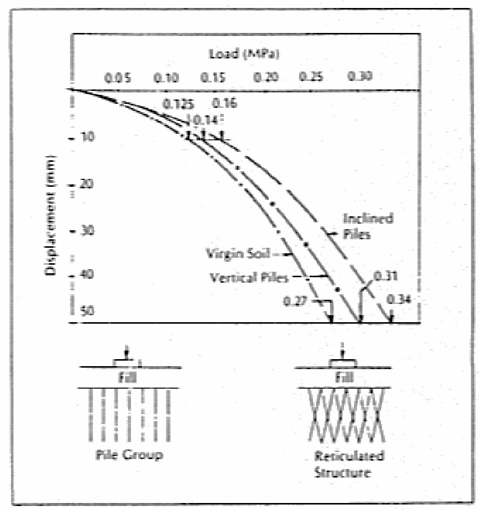


Figure 6 Field test data for different pin pile arrangements. (Plumelle, 1984)

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, 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, Plumelles's (1987) full-scale testing yielded the results shown in Figure 6 that confirmed the trends of Lizzi's earlier model tests (Figure 7). 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.

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, developments continue to be made. These have been directed towards providing piles of superior performance more cheaply and in more challenging structural, geological, or environmental settings. Three groups merit special attention:

a) Post-grouting of bond zone
b) Reinforcement of free zone
c) Preloading

By injecting cement grouts into the bond zone after the initial 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 as in the GEWI pile system) (Herbst, 1982) 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

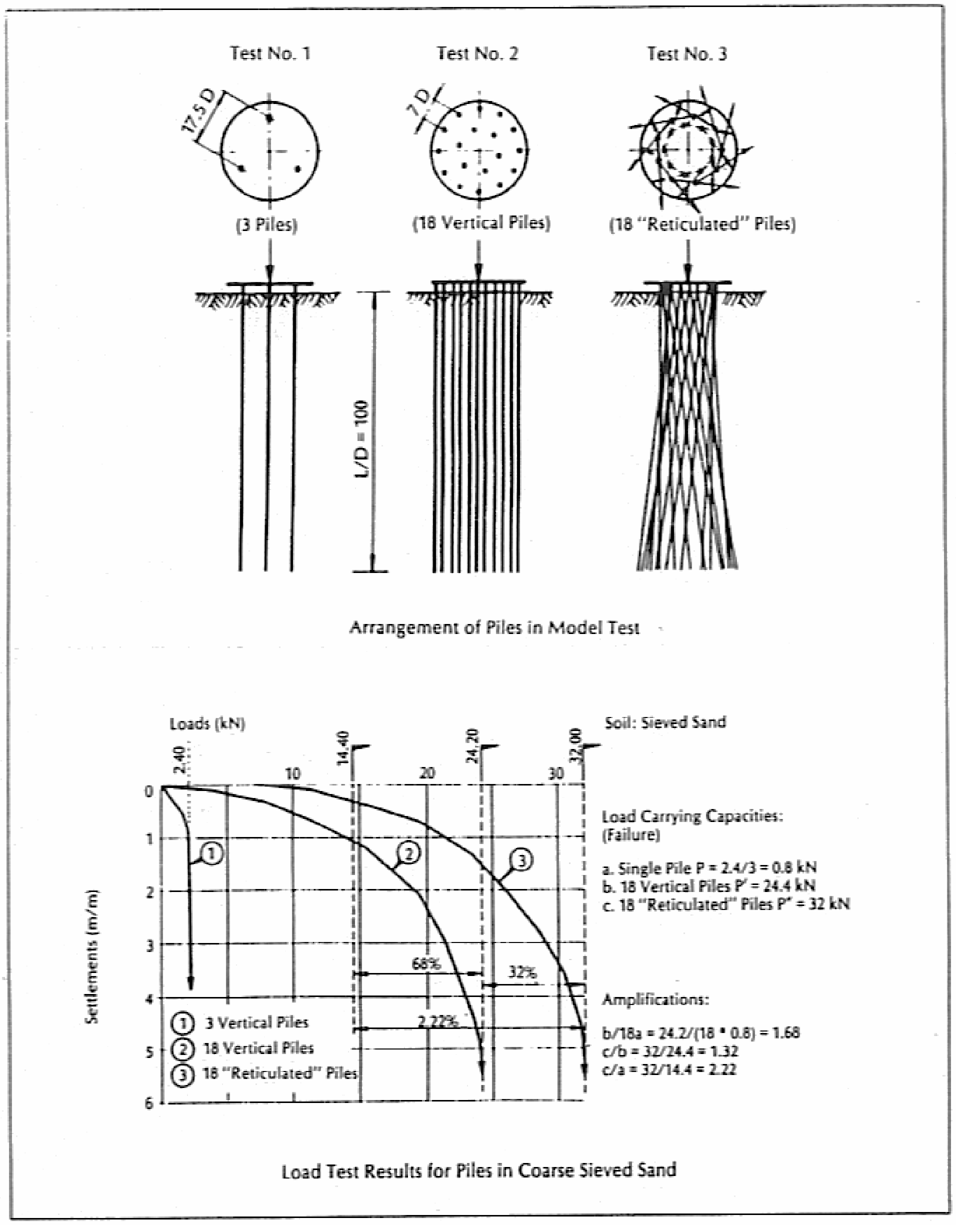


Figure 7 Model test data for different pin pile arrangements in coarse seived sand. (Lizzi, 1978)

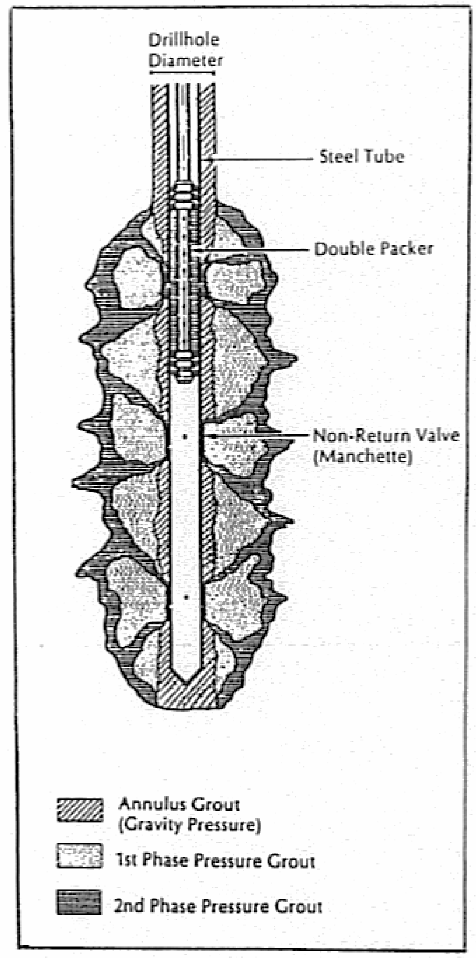


Figure 8 Concept of repeated post-grouting in increasing effective pile diameter. (Mascardi, 1982)

pipe and grout is ejected through the rubber sleeved ports at regular intervals (see Figure 8).
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 Mpa are most commonly used.

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 displacements 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 favourable response to post-grouting in stiff clay.
No experience of good behaviour in very soft non-consolidated clay or soft peat has been recorded. A major contract is currently underway in New York wherein the high capacities of the minipiles to be used in an elevated highway refurbishment can only be achieved through systematic post grouting of the founding fine-medium sands. This will probably be the forerunner of many post grouted minipile jobs in the Eastern States as it is a very "high profile" contract which has elicited great interest amongst contractors and consultants alike.

Reinforcement of at least the upper part of the pile to guard against buckling or bursting is becoming more common. In this case, the drill casing may be pushed back down into the bond zone (3 to 6 m) after the completion of initial 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/movement performance as also exceptional, as illustrated in Figure 9, while additional resistance is automatically provided against lateral loading.

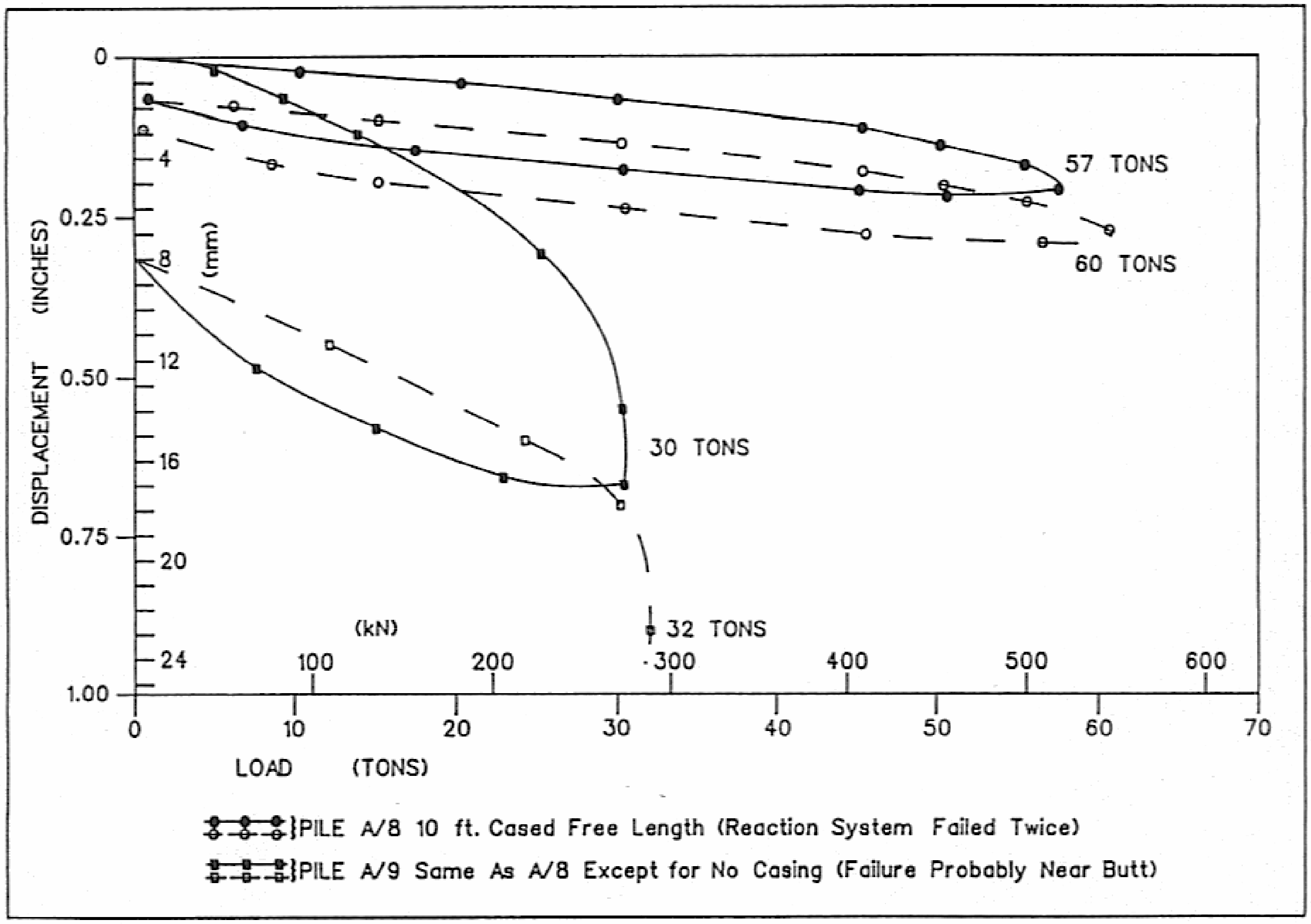


Figure 9 Pin pile load test data from a site in Coney Island, NY, showing the benefits of a steel liner through upper strata above the bond zone.

Although pin piles furnish excellent load/movement characteristics, with total deflections in normal cases being less than 1 cm at service load, there are cases where even this magnitude of movement is unacceptable to a particularly delicate structure. Preloading has been used, wherein the pile has had the service 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 at the head. It is preloaded by a hydraulic jack, acting on the anchor pipe and locked off against the pipe when the desired load or movement is achieved. An alternative system to preload pin piles was used, at the project at Warwick, NY, 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 has been found to be 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 movements 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, service loads of up to 50 to 60 tons can be generated safely, with far higher individual capacities recently recorded when founded in rock (Table 1). For very heavily loaded structures (e.g. bridge piers), groups of piles are usually required. Even here, however, it would seem that the resulting group effect is positive, as opposed to demanding reduced individual loadings as is the case with conventional large diameter piles.

Testing is relatively economical, 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.

A 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. A second drawback in the States is their relative novelty, which occasionally prevents clients "risking" the technique, mindful of the zeal for litigation shown in certain quarters.

Nevertheless, pin piling is becoming increasingly popular (Bruce and Nicholson 1988) to the extent that in some of the older Eastern cities, there is arguably a greater intensity of activity in this field than anywhere else in the world.

3. INSITU REINFORCING

Three basic classes of in situ reinforcement can be recognized (Figure 10):

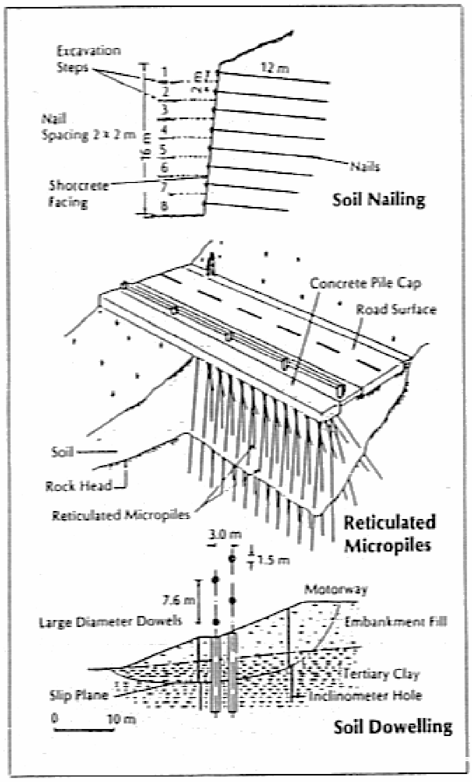


Figure 10 The family of in situ soil reinforcement techniques. (Bruce and Jewell, 1986)

a) 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
b) 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, holding back the soil behind.
c) 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 mini 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.

3.1. Evolution and Applications

Early applications of small diameter inserts 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 executed in Europe and being considered in the U.S. for Metro work 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 the Dartford Tunnel Approach Road 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), higher capacity tubular mini piles (40 tons service load) 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 cantilever performance.

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 1.0 Mpa. 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 complete 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 if 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, however, 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:

Table 2 Summary of reticulated micropile wall case histories. (US Applications 1977–1989, in date order)

Project Name and Location	Ground Conditions	Upslope/ Downslope Grade (U/D)	Depth to Slip-Surface at Wall (ft)	Cap Beam Geometry LxWxH (ft)	PILE DATA Diameter (in.)	Reinforcement Type and 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	Micaceous phillite ϕ=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 gauge data indicate mostly compressive strains except near the pile cap
Route 23A Catskill State Pk. New York	Moist very dense glacial till with boulders ϕ=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	Random fill colluvium and weak red shales ϕ=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	Random fill underlain by wet sandy clay & a hard clayey residual bedrock ϕ=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
SR 4023 Armstrong County Pennsylvania	Random fill underlain by colluvial clay & shale ϕ=17°, c=0 γ=120 pcf	Flat Roadway/ 1.75H:1V	45 for 140' and 35 for 180'	320x4.5x3.0	6	#9,#14, and #18 Rebar. Grade 60	1.25 for 140' and 0.9 for 180'	22 0 and 26 0	57 and 60	10-15 for both	20 for both	N.A. under construction	Design probably overconservative due to local Code constraints

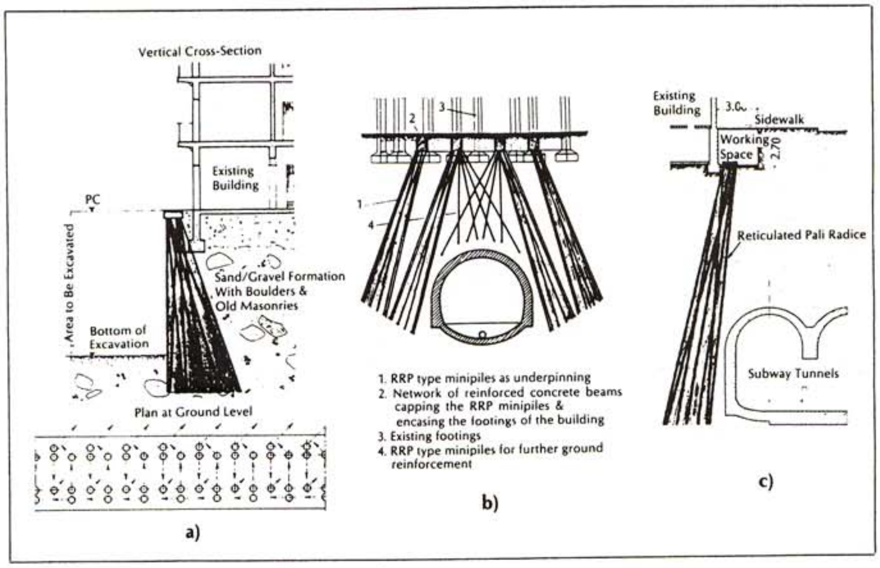
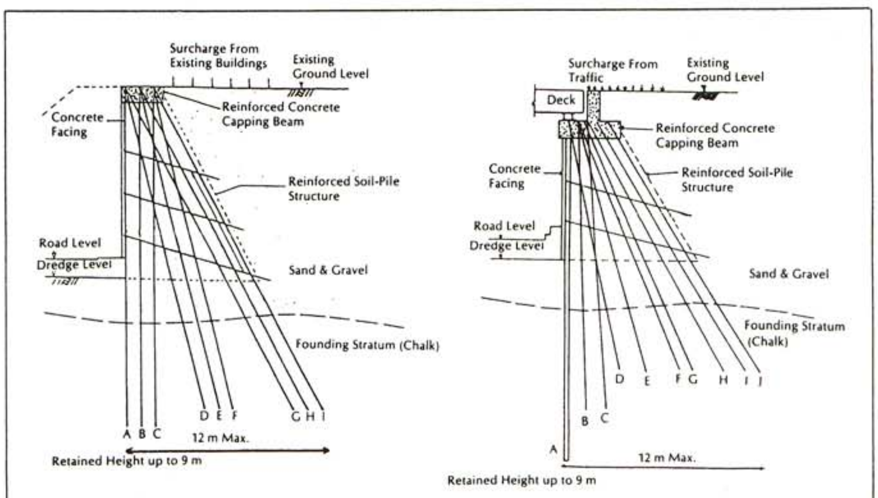


Figure 11 Applications of reticulated micropiles used as in situ reinforcement a) for cut and cover excavation, and b) around bored tunnels. (After Lizzi, 1982)



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)

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

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.

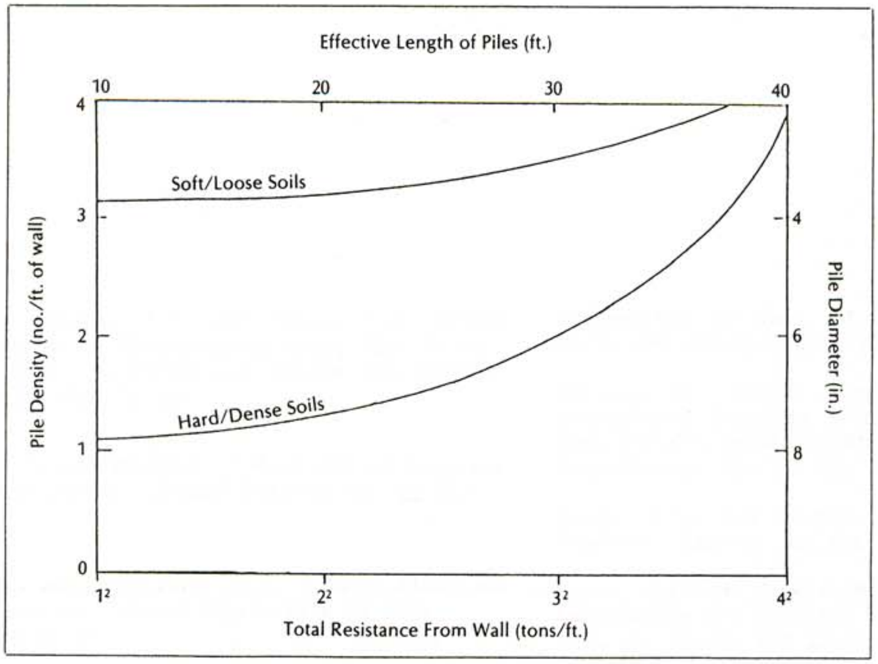


Figure 13 Chart for the preliminary design of reticulated micropile walls. (Dash 1987)

As an example, the general design for the Mendocino wall referred to in Table 2 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 inserts 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 (Figure 13).

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 high 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 inserts deemed necessary. These two consequences should result in a greater acceptability of the technique in the U.S.

4. CONCLUDING REMARKS

Load bearing minipiles, and similar 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 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.

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