Minipiles were born in Europe, but they're spending their prime years solving some of America's trickiest structural problems.

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MINIPILES MATURE IN AMERICA

We hear a great deal about the tide of European technologies lapping at U.S. shores. One such technology, minipiling, was born in Europe 40 years ago but is reaching maturity in the United States.

Minipiles have many names—pin piles, micropiles, root piling, pali radice, pieu racine, Wurzelpfahle and Estacia Raiz—but all are essentially small diameter drilled and grouted piles. Their relatively small diameter, rarely larger than 12 in., is clearly a distinctive feature. But their fundamental characteristic is their ability to be constructed with equipment used for anchoring and grouting, unlike conventional piles that need to be driven or bored.

Minipiles can be constructed to considerable depths, through all types of soil, rock and obstructions, and in virtually any inclination. They have a high slenderness ratio and transfer loads almost entirely by shaft friction, eliminating any requirement for underreaming at the base. All minipiles feature substantial steel reinforcing elements, so they can sustain axial loading in both tension and compression. The reinforcement can also be designed to resist bending stresses safely and with minimal displacement.

Constructing with minipiles minimizes vibrations, ground disturbance and noise, and they can be installed under difficult working conditions. Though they may be more expensive than conventional driven or large diameter piles, minipiles may be the only guaranteed solution when given a particular set of ground, site access, environmental and performance conditions.

THE ONLY GAME IN TOWN

Minipiles have a relatively high load carrying capacity (for their diameter) and very small settlements. Piles installed wholly in soils can provide safe working loads approaching 100 tons. When founded in rock, they can sustain safe working loads of as much as 300 tons.

Load holding capacity can be improved substantially by postgrouting—the injection of secondary cement grout at high pressures after the initial grout has set. Settlements to structures being underpinned can almost be eliminated by preloading the piles to the working load and by prestressing, so that no further pile movement occurs when the structural load is finally applied.

Minipiles have gained popularity in this country because there is a growing trend towards remedial and rebuilding work in our older cities and industrial centers. As a result, foundations have to be upgraded or replaced to resist new increased loadings, or have to be protected from settlements caused as a result of nearby construction of tunnels or other deep excavations. Under such circumstances, there may be major practical restrictions on the options available to the engineer. There is probably a greater use of minipiles in Boston than anywhere in the world.

For instance, the properties at 739-749 Boylston Street in the Back Bay area of Boston were completed in 1906. Until recently the buildings were derelict, but in the early 1980s, the Pilgrim Trust Company acquired them for redeveloping and refurbishing. The building at 739 Boylston Street is a six story structure that will have retail space on the basement and first floors, office space to the eighth floor and a mechanical penthouse level above.

The structure was originally constructed on pile caps bearing on timber piles. Increased loadings from the new construction made additional support necessary under enlarged pile caps.

Piling had to be installed inside a partially demolished basement, about 10 ft below the existing sidewalk, giving a minimum of 8 ft of headroom. The difficult and restricted access ruled out the use of conventional piles. Based on a cost and performance analysis, the

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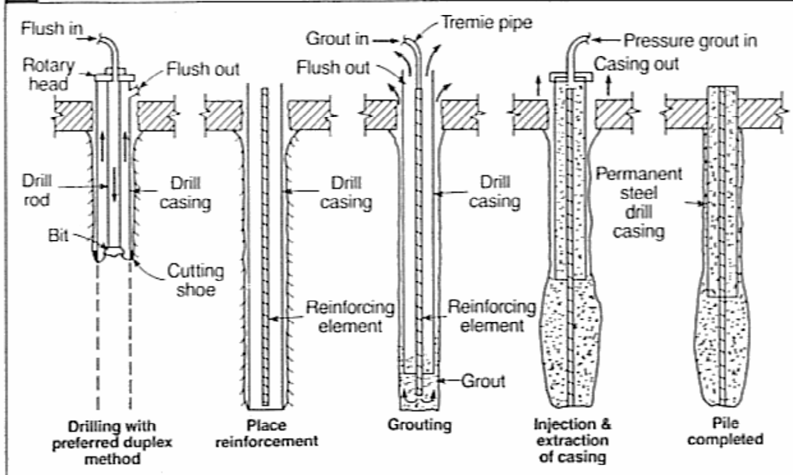
FIGURE 1.
CONSTRUCTION STAGES



Stages of Boston pin pile construction.

owner accepted an alternative bid: install about 260 minipiles with working loads of 40 tons in compression and 12 tons in tension. In works of this type, with these restrictions, prices of $55 per foot can be expected. Admittedly, that amounts to much more per foot than conventional piling systems, even if they could be used. But the prices are deceptive. Pin piles provide more support per foot, so fewer feet are needed.

The fill consisted of saturated loose gray-brown fine sand and silt, and overlaid soft gray organic silt with traces of shells, sand and gravel. The founding layer occurred at about −4 ft and was 18 ft to 24 ft thick throughout the site. It consisted of medium dense/dense fine-medium sand with a trace of silt. The pile lengths were kept within this horizon so as not to penetrate the weaker underlying Boston blue clay.

The piles were designed on the basis of an ultimate load 2.3 times greater than the design working load: 92 tons in compression and 27 tons in tension. The length of the load transfer zone was designed on the basis of analogous soil anchor experience. Calculations showed that an ultimate load of 92 tons required a load transfer length of 15.5 ft.

Further computations showed that the use of 5.5 in. steel casings with a wall thickness of 0.362 in. and a minimum specified yield stress of 55 ksi as the major load bearing element was safe. In addition, an internal 1 in. dia., 60 ksi rebar adequately transferred loads in the founding horizon.

A diesel hydraulic trackrig installed the piles. Water flushed the 5.5 in. casings to about 8 ft below the surface before the casings were pushed to locate the top of the dense bearing strata. Rotary drilling then resumed in the sand to full depth. Neat Type I grout with a water cement ratio of about 0.5 was placed by tremie, followed by the rebar. Pressure grouting of the sand was carried to a minimum of 60 psi during extraction of the casing for the 15 ft or 16 ft bond zone. The permanent steel casing was then pushed back down 3 ft to 5 ft into this pressure grouted zone and left in place.

Grout takes generally ranged from 2.5 to 3.5 times nominal hole volume, confirming that the enhanced effective diameter of the bond zone had been achieved. Grout cubes at 14 days gave unconfined crushing strengths of over 6,000 psi.

TESTING AND PERFORMANCE

Prior to the full-scale piling program, two typical piles were subjected to compressive and tensile load testing. A telltale anchored near the tip and an outer steel liner placed around the 5.5 in. casing above the bond zone prevented any load transfer in the upper soil. Ground anchors provided the reaction for each pile. The elastic 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 fell well below the level recommended in the local building code. The performance in tension was equally satisfactory.

Overall, four piles had to be replaced due to construction problems. In addition, two piles had to be added to the design, bringing the contract total to 262, averaged in groups of 4–6 piles per cap.

After a three month period of pin pile installation (fig. 1), it took about eight months to complete the major structural rebuilding work. Since the job was completed in May 1987, the piles have performed flawlessly.

VIEW FROM THE BRIDGE

Pin piles are also holding up well while holding up part of the I-78 dual highway, which crosses the Delaware River between Pennsylvania and New Jersey (fig. 2). The highway rests on seven span, multigirder bridges. Generally, foundations on the Pennsylvania side incorporated driven H piles, whereas the river piers and the New Jersey piers were to be founded on solid rock. Or so it was thought.

Excavation to the planned elevation for the footing of Pier E-6 on the eastbound structure revealed that the rock was nonexistent. Another 15–20 ft of digging showed only random rock thicknesses of several feet and a highly pinnacled, irregular bedrock surface. The excavation was filled with lean mix concrete, while the foundation design was reconsidered.

Enlarged spread footings, H piles in predrilled holes and the complete elimination of the pier proved infeasible. So two other alternatives were considered: the installation of six 36 in. caissons, each with a working load 360 tons; and the placement of 24 minipiles, each of nominal working load 100 tons.

COST EFFECTIVE

The costs of the large diameter caissons are difficult to estimate because their design and construction would have varied widely as different soil and rock conditions were encountered. Because the geology of the area was so disparate, construction might have been delayed dramatically by installation problems.

The pin piles cost in excess of $100 per foot. Though expensive, they did offer a number of advantages that made their use attractive and cost effective. Because pin piles transfer their loads by skin friction, not end bearing, there was no possibility that the pile could fail by punching through into a soft underbed immediately under the founding level. Even the diverse soil conditions found in this site would not dramatically alter construction times. In addition, test piles could prove their effectiveness in advance.

The bedrock was a Cambro-Ordovician dolomite limestone referred to locally as the Allentown Limestone. It proved to be moderately to highly fissured, cherty and very susceptible to karstic weathering. Major clay filled beds intersected the bedrock, even at depths greater than 100 ft. The rock mass proved highly variable laterally and vertically.

The owner's design regulations permitted a maximum average rock/grout bond at working load (100 tons) of 50 psi and a maximum allowable reinforcement steel stress at working load equivalent to 45% of yield.

These factors led to the selection of a load transfer zone with a diameter of 8.5 in. and a 15 ft length in competent rock. The major structural component of these pin piles was a 55 ksi low alloy steel pipe 7 ft in o.d. and a 0.408 in. wall thickness.

A separate test pile, 30 ft long with only 5.33 ft of bond, was load tested to 205 tons, using rock anchors as reaction. This particular socket length for the test pile was selected because its average grout-rock bond would be 304 psi, and the typical grout-steel bond would be 250 psi. Both met or exceeded the ultimate values. An outer sleeve of PVC pipe extending to the top of the rock socket ensured load transfer only in the socket. And a 6 in. thick wooden plug attached to the bottom of the steel pipe ensured that no load could be transferred in the end bearing.

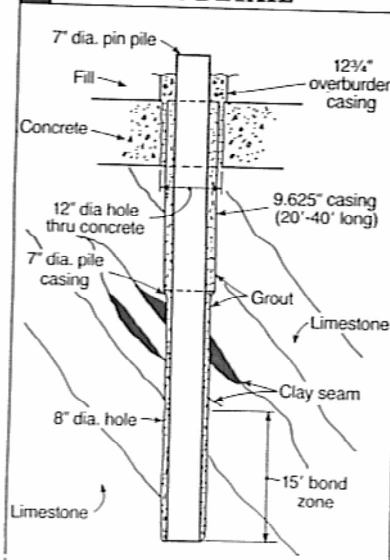
Total settlements of 0.367 in. and 0.373 in. were recorded at each successive cycle to 205 tons. Creep of 0.011 in. was recorded over one hour at these loads. The permanent set after this operation was 0.07 in.

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

Seven of the piles were required to have continuous 15 ft bond zones. But because the rock was likely to be variable, the 15 ft bond zone did not have to be continuous for the rest of the piles. They did, nevertheless, have to meet certain other restrictions. The lower part of the zone had to contain at least 10 ft of continuous sound rock. Soft interbeds had to be less than 3 in. thick. A zone of acceptable load bearing rock had to be at least 5 ft. thick. Interbeds within the overall bond zone had to be regrouted and redrilled.

As in the Boston project, track-rigs drilled all the holes. First the contractor installed a 10.75 in. o.d. casing through gravel and into the concrete backfill. A 10 in. dia. down-the-hole hammer drilled through the concrete and 9.518 in. steel casings were installed through the less competent upper horizons. After these were surveyed for plumbness and then grouted in place, an 8.5 in. hole was drilled to final depth to ensure a minimum 15 ft bond zone. The holes were flushed clean and a 7 in. o.d. pipe was installed, then surveyed to make sure it was vertical with no more than a 2% deviation. Finally, it was grouted at 50 psi.

FIGURE 2.
PIN PILE DETAIL



I-78, New Jersey.

PROBLEMS

Throughout the construction, the very adverse geologic conditions posed major drilling problems. These were resolved, at length, by repeated pregrouting and redrilling.

In addition, during installation of the reinforcing pipe in the last and deepest pile, a thread parted and a 130 ft length of pipe fell into the 200 ft deep hole. Borehole TV revealed the casing to be further ruptured 30 ft above the bottom of the hole, due to its impact with the bottom. After various attempts at recovery and recoupling, it was decided to grout the pile, having suspended a 20 ft long, 4.5 in. dia. 150 ksi steel pin with centralizers from 62 ft to 82 ft below the top. The pin was placed to ensure effective load transfer across the upper discontinuity. A very rigorous extended load test was then executed to 170 tons. The performance of the pile proved excellent.

Overall, the total drilled length of 1,920 linear ft corresponds with the total foreseen quantity of 1,710 linear ft. However, the lengths of individual piles ranged from 43 ft less to 30 ft more than projections. A volume of grout equivalent to eight times the nominal hole volume drilled was injected. Much of this was consumed in the zone above the rockhead during pregrouting operations.

The bridge is now complete, and the performance of the pin pile pier has proved exceptional.

CREDITS

At Boston, the owner was Pilgrim Management Trust, the engineer was Goldberg Zoino & Associates, Newton Upper Falls, Mass. and the general contractor was Perini Corporation of Framingham, Mass. PennDOT was the owner of the Warren County bridge project. Modjeski & Masters was the engineer for the project and the general contractor was GA&FE Wagman.

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