Nicholson Construction Company
12 McClane Street
Cuddy, PA 15031
Telephone: 412-221-4500
Facsimile: 412-221-3127

Pin Piles in Karst Topography

by

F. Tarquinio, P.E.
Nicholson Construction Company, Cuddy, Pennsylvania

S. Pearlman, P.E.
Nicholson Construction Company, Cuddy, Pennsylvania

Presented at:
Sinkholes and the Engineering and Environmental Impacts of Karst
8th Multidisciplinary Conference
Louisville, Kentucky
April 1-4, 2001

01-01-130

Fred S. Tarquinio, P.E & Seth L. Pearlman, P.E., Nicholson Construction Co., U.S.A., info@nicholson-rodio.com

ABSTRACT

Karst topography is a problematic geology located throughout the United States. It is difficult because it is highly unpredictable and variable in nature (see Figure 1), since the mineral composition of the bedrock (limestone) is vulnerable to the formation of solution cavities and sinkholes. The upper layers of the limestone are typically polluted with voids, massive clay seams and artesian conditions. The overburden soils above the limestone are typically silts and clays that are susceptible to large settlements when heavy foundation loads are placed on them. For heavily loaded structures or structures that are sensitive to settlement, deep foundations are necessary. Typical deep foundations are drilled shafts or driven piles. These foundation types transfer the foundation loads to the refusal material or bedrock. However, in the karstic formations, the refusal material may be underlain by voids and clay seams. Therefore, foundations bearing above these problem zones may be unstable. The solution to this problem is the use of Pin Piles. Pin Piles are drilled and grouted piles typically ranging from 5 to 12 inches in diameter. Their capacity in soil and rock is derived entirely from friction. In karst areas where several strata of bedrock must be penetrated, the Pin Piles are readily advanced through the hard strata and socketed into competent bedrock. The piles are drilled with an eccentric down the hole hammer that advances the casing as the hole is being drilled. The hammer reacts to hard drilling in a way that is both visible and audible; therefore verifying the competency of the rock.

Figure 1 – Karst Uncovered

INTRODUCTION

Many cases exist where it is not practical or economical to install more traditional deep foundation elements such as driven piles or drilled shafts. In these cases, such as in ground where obstructions, boulders and solution cavities are present, Pin Piles have been selected. Pin Piles are drilled-in elements typically ranging from 5 to 12 inches in diameter, which consist of steel casing, steel reinforcement, and cement grout. They derive capacity in the ground from side friction and work equally well in both compression and tension.

These piles were developed in Europe in a simpler form, typically using only a central rebar core encased with cement grout placed into a small diameter drilled hole. These minipiles or micropiles were installed as individual elements or groups with cumulative benefit.

In the United States, particularly when considered for building applications, it was realized that traditional mini or micro piles were limited by their structural capacity. That is, the rebar core with virtually unconfined grout surrounding it was not very resistant to high compressive loads or any manner of lateral bending or eccentric loading.

Since these elements are most often installed using a steel drill casing, it was the innovation of the American contracting community to incorporate the steel casing into the pile designs. This ductile steel casing pile, Pin Pile, provides a high degree of structural resistance in the soft upper soils, and allows for the full optimization of the underlying competent geology. It is often the physical constraints that act as a trigger for use of Pin Pile technology. Such situations may be:

- subsurface obstructions or difficult ground,
- limited overhead clearance,
- vibration or noise sensitivity,
- settlement sensitivity,
- limited plan access,
- the need to install elements in close proximity to or through existing footings, columns, walls, or other structures.

Geotechnical complications may act as a single driving force for Pin Pile selection, or may complement the already difficult physical limitations of a project. The geotechnical situations other than what might be called "conventional" which Pin Piles may be conveniently installed are:

- karstic limestone geology (that includes voids or soil-filled solution cavities),
- bouldery ground or glacial till,
- variable and/or random fill,
- underlying existing foundations or man-made obstructions,
- rock formations with variable weathering,
- or soils under a high water table.

GEOTECHNICAL ASPECTS

Pin Piles derive their load carrying capacity from side friction in suitable ground stratum, either soil or rock. Ground types that are capable include:

- stiff or hard non-plastic clays or silts,
- sands and gravels,
- rock formations,

- combination materials such as glacial tills or residual soil formations with variably weathered rock inclusions.

Since the primary load carrying capacity in the ground is derived from frictional bond in soil or rock, the Pin Piles can develop high capacity in both compression and tension. In compression, Pin Piles typically range in working load from 50 to 200 tons. In tension, their capacity is nearly identical geotechnically, and is primarily limited by the amount and detailing of the core reinforcement and casing joints used in the elements.

When subjected to lateral loadings, the piles derive resistance from the horizontal response of the adjacent soils and can sustain significant lateral deflection within the available structural pile capacity. Seven-inch diameter Pin Piles have been laterally tested in variable urban fills, with a free head condition, to up to 19 kips with 0.75 inch deflection. On a site with stiff alluvial soils, in a free head condition, a seven-inch Pin Pile was tested to 24 kips with 0.3 inch deflection. Where significant bending capacity is desired due to lateral loads, a larger upper casing can be installed to provide a pile with a two-part cross-section.

INSTALLATION METHODS

The ability to install Pin Piles in the most difficult and problematic geotechnical situations is a major advantage in their use for building foundations. This capability is gained principally by the optimal selection of drilling and grouting techniques. Through proper selection and experienced execution, good results are obtained.

Pin Piles are installed using rotary drilling techniques similar to those used in the oil and gas industry. The piles develop their geotechnical capacity through grout to ground adhesion in the bond zone. In soils this bond is typically developed using pressure grouting, and in rock, tremie grouting. The principle type of drilling and grouting techniques used by the writers in karstic areas is described below.

Rotary Eccentric Percussive Duplex Drilling

This method uses an inner rod and an outer casing, with the spoils flushed inside the casing. The bit on the inner drill rod is equipped with a down-the-hole hammer. The hammer bit is specially designed to open up during drilling to a diameter slightly larger than the outside diameter of the drill casing (see Figure 2). This bit provides a slightly oversized hole through obstructions or rock and thereby allows the casing to simultaneously follow it down. Compressed air is used to drive the hammer and also acts as the drilling fluid to lift the cuttings. This drilling method is used in soils containing large amounts of obstructions such as cobbles, boulders or demolition waste and is also very effective in advancing a drill casing through highly fractured rock zones in karst. This method is most effective for installation in karst.

Figure 2 – Eccentric Hammer Inside the Casing

Tremie Grouting

This is a method used to place grout in a wet hole. A grout tube is lowered to the bottom of the drill casing or rock socket. Grout is pumped through the tube as it is slowly removed from the hole. As the grout fills the drill casing or hole, it displaces the drilling fluid. Tremie grouting is primarily used where the Pin Pile bond zone is founded in rock. When working in highly broken or fractured rock or in voided, karstic situations, grout loss is possible and may warrant testing for a sealed bond zone. When this is done it is typical to perform water testing and seal grouting as required, then re-drill, and test again. This potentially repetitive process requires commercial compensation using unit prices for these variable and unpredictable quantities.

Installation Sequence

The response of the down the hole hammer indicates whether or not the rock of sufficient quality is penetrated. Once a competent bond zone is established, the casing is withdrawn to the top of the bond zone and the pile is filled internally with grout. Once the grout level has stabilized in the bond zone, the centralized reinforcing steel is placed. A typical cross-section is shown below (Figure 3).

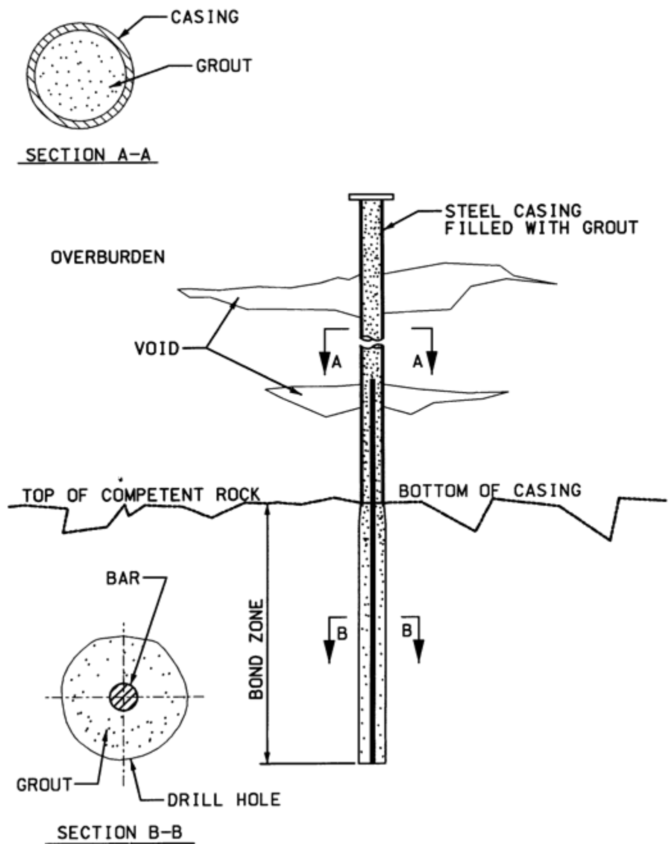

Figure 3 – Typical Cross-Section

DESIGN

Materials

Pin Piles are typically constructed using steel casing with special machined flush jointed threads. The casing meets the physical properties of ASTM A-252 Grade 3, except that the minimum yield strength is typically 80 ksi. This material is most often mill secondary API drill casing. As such, material certifications are not available. The physical properties are confirmed by cut coupons from representative pieces of casing. The core reinforcing steel is Grade 60, 75 or 80 reinforcing bar (ASTM A615, A616 or A617) or Grade 150 prestressing bar (ASTM A722).

The grout typically consists of neat cement and water mixed with a high shear colloidal type mixer. This grout has a fluid consistency, a water/cement ratio of about 0.45, and a typical minimum unconfined compressive strength (from cubes) of 4,000 psi in 28 days.

Structural Capacity

The structural design of Pin Piles for building foundations is typically not found in local building codes. However, based on our experience in conjunction with some applicable codes, the following equation is used.

$$P_{all} = (0.40 \text{ to } 0.50)F_yA_s + (0.35 \text{ to } 0.45)f'_cA_c$$

where: F_y = Yield Stress of casing or rebar
A_s = Cross-sectional area of casing or rebar
f'_c = Unconfined Compressive Strength of the grout at 28 days
A_c = Cross-sectional area of the grout

The most applicable factors are selected upon the experience of the contractor, the experience of the structural engineer, and the observed performance of the test piles. The equation above is conservative as long as the pile is structurally configured to take advantage of confinement. That is, the grout needs to be confined inside the pipe casing, and in the rock bond zone, the grout receives lateral confinement from the ground. This effect has been confirmed many times through load test performance at ultimate loading, with stresses far in excess of typical values. More research and modeling is needed in this area to further quantify this benefit so that further economy of design is possible.

Geotechnical Capacity

The bond length is determined by experience and by previous load tests in the rock. A typical allowable bond stress in karstic bedrock with unconfined compressive strengths ranging from 15,000 to 20,000 psi is 100 psi. The bond zone capacity is calculated as a typical friction pile. Tip resistance is usually neglected. The following calculation is used.

$$P_{all} = \sigma\pi dL$$

where: σ = Allowable bond stress of competent rock in bond zone
d = Diameter of bond zone
L = Length of bond zone

Testing

All projects of any significance justify full scale testing (ASTM D1143) of at least one pile unless there is significant confidence and prior experience with the founding stratum. The purpose of the testing is to verify both the geotechnical capacity of the bond zone and the structural performance of the pile.

CASE HISTORIES

Exton Mall and Garage, Exton, Pennsylvania

Exton is a city located approximately 20 miles west of Philadelphia. The owner of the Exton Square Mall was constructing a second-story addition and a new parking garage. The initial foundation contractor encountered difficulty in drilling through the karstic limestone underlying the site. This contractor's method of installation resulted in several pile failures during the load test program. Nicholson was called in on short notice to take over the construction. The construction for the addition to the mall was completed in two areas: drilling from inside the mall itself, and drilling at close proximity to the perimeter of the building. The work on the inside of the mall took place during hours when the mall was non-operational. This meant construction crews would work night shift, ensuring the stores were cleaned and functional for normal business during the day. Special piping was used to remove the spoils from the drilling up through the roof of the mall, and down into refuse containers on the ground. Drilling conditions inside the mall involved tight access and limited headroom (12 feet). Crews moved merchandise and protected it where necessary.

Bedrock at the site consisted of karstic limestone with voids and clay seams. The top of competent bedrock ranged from 20 feet to 150 feet below the existing ground surface. The maximum design working load was 300 kips in compression. A total of 294 Pin Piles were installed in the interior of the mall and 111 Pin Piles were installed around the perimeter. The average pile lengths were approximately 34 feet, ranging from 20 to 150 feet below the existing slab elevation. A total of 355 piles were installed for the new parking garage, with pile lengths averaging 43 feet, ranging from 25 to 85 feet. The Pin Piles consisted of a 7 inch O.D. by 0.50 inch wall outer steel casing above the competent rock bond zone and two #18 Grade 75 all-thread bars from the bottom of the 10-foot bond zone overlapping 5 feet into the steel casing. The casing had physical properties equal to or exceeding ASTM A252, Grade 3, except that the minimum yield strength was equal to or greater than 80 ksi. Physical properties of the steel casing were determined by coupon tests on random samples. Pile grout consisted of Type I Portland cement grout (w/c = 0.45), with a nominal 28 day strength of 4,000 psi.

The piles outside the existing mall were installed with a large track-mounted Casagrande C12 drill rig, and the piles inside the facility were installed with both Davey Kent DK-50 or Klemm mini drill rigs. All drills utilized rotary eccentric percussive drilling techniques. For the outside piles, the casing was advanced in 20 foot, flush joint threaded sections to the bottom of the bond zone. Piles drilled inside were advanced with 5 foot, flush joint threaded casing due to overhead limitations. Once the bottom of the bond zone was established, the casing in all piles was retracted to the top of bond zone elevation. Consolidation grouting of the bond zone was then performed using a 1.5-inch diameter steel tremie tube.

Three production Pin Piles installed were tested in accordance with the Standard Test Method for Piles Under Static Axial Compressive Load (ASTM D1143-81, Reapproved 1994). In particular, the Quick Load Test Method for Individual Piles (Section 5.6) was used. The maximum test load applied was 600 kips equal to twice the design load.

Beaver Stadium Expansion, Penn State University, State College, Pennsylvania

The Beaver Stadium Expansion for Penn State University in State College, Pennsylvania is currently under construction. When finished, the expansion will add approximately 10,000 new seats, including new luxury boxes. Also, a new scoreboard was constructed. Since the expansion ties in to the existing stadium, the new structures are very sensitive to differential settlements.

The project site is underlain by karstic geology, consisting of very hard dolomitic limestone (unconfined compressive strengths over 20,000 psi). The hard bedrock is pinnacled, with almost vertical bedding planes. This geologic formation is prone to voids, clay seams, varying water levels and greatly varying overburden depths. The top of bedrock at the project site ranged from about 5 feet to over 75 feet below the floor slab elevation.

Two pile designs were used, supporting both new interior and exterior columns. The exterior piles were designed to support loads of 150 tons in compression and 75 tons in tension. The interior piles were designed to carry 50 tons in compression. All of the piles were installed vertically and consisted of three distinct components. The first was the outer casing that provided additional bending capacity and lateral stiffness. The second component was the centralized reinforcement bar that extended to the tip of the pile and overlapped inside the outer casing. The third component was the grout which was tremied into the pile.

Exterior piles consisted of a 7-inch O.D. x 0.5 inch wall thickness threaded flush joint steel drill casing with a minimum yield of 80 ksi. The interior piles consisted of 5-1/2 inch O.D. x 0.415 inch wall casing. The bond zone section was made up of either one #18 Grade 75 rebar (50 ton pile) or two #18 Grade 75 rebar (150 ton pile) in a 5 to 10 foot long tremie grouted bulb with the bar overlapping 5 feet into the casing. The neat cement grout had a minimum 28-day strength of 4,500 psi. The casing was made from mill secondary API drill casing. The piles were installed using rotary eccentric percussive duplex drilling with air as the flushing medium.

Three compression and tension pile load tests were conducted on piles with 150 ton compression capacity and 75 ton tension capacity to verify the adequacy of the pile design and installation methods. To be acceptable, a test pile had to carry a test load equal to two times the design load and less than 0.5 inches deflection. The Beaver Stadium Expansion project involved installing over 600 Pin Piles. The Pin Piles were successfully installed despite drilling in the very difficult karst geology that included pinnacled limestone, clayseams, voids and solution cavities.

ACKNOWLEDGMENTS

The authors would like to thank the numerous owners and design consultants who provided the forward thinking by allowing the use of this technology in their projects. We also wish to thank other employees of Nicholson Construction for their time.

REFERENCES

Kenny, J. R., "Behavior and Strength of Composite High Strength Steel Tubular Columns", Thesis for Master of Science, University of Pittsburgh, 1992

Groneck, P.B., Bruce, D.A., Greenman, J.H., and Bingham, G., "Pin Piles Save Silos", Civil Engineering Magazine, September 1993

Munfakh, G.A., Soliman, N.N., "Back on Track at Coney Island", Civil Engineering Magazine, December 1987

Morschauser, G.B., Davis, J.E.B., "Replacing an Urban Foundation", Civil Engineering Magazine, December 1990

Pearlman, S.L., and Wolosick, J.R., "Pin Piles for Bridge Foundations", Proceedings of the 9th Annual International Bridge Conference, June 1992, pp. 247 - 254, #IBC-92-40.

Pearlman, S.L., Richards, T.D., Wise, J.D., and Vodde, W.F., "Pin Piles for Bridge Foundations: A Five Year Update", Proceedings of the 14th Annual International Bridge Conference, June 1997, pp. 472-480, #IBC-97-53

Pearlman, S.L., "Pin Piles for Structural Underpinning", Proceedings of the 25th Annual Deep Foundations Institute Meeting, October 2000

Tarquinio, F.S. and Pearlman, S.L., "Pin Piles for Building Foundations", Proceedings of the 7th Annual Great Lakes Geotechnical and Geoenvironmental Conference, May 1999