High Capacity Micropiles In Karst:

Challenges and Opportunities

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Abstract

Extreme variations in ground conditions which categorize karstic limestone terrain inevitably create challenges during design and installation of deep foundation systems. For example, the elevation of rock head may vary greatly over short distances, while for substantial depths below rock head, one may anticipate the presence of major solution features. Such features may be entirely open, or may be partially or completely filled with products of the limestone degradation. High capacity micropiles of the type described by FHWA (1997) have, for some years now, proved to be a technically and economically viable deep foundation system in such terrain. Frequently, they have been the only plausible solution. This adaptability is a result of the large variety of small diameter borehole drilling techniques which can be exploited to ensure penetration in such conditions, and also the fact that micropiles transfer load by skin friction. The design implication is that if certain thickness of "good" rock is encountered, then a certain pile capacity can be guaranteed. There is therefore dramatically reduced risk of piles "punching through" into a solution feature as would occur in the case of an end-bearing pile. During the last several years, the authors have been involved in the design, construction and performance testing of high capacity micropiles in karst. This paper provides an overview of these experiences and provides recommendations as to most appropriate practice. Reference is made to several recent case histories.

Introduction

Most recent domestic studies describing micropiles relate to elements which are small diameter ($\leq$12 inches), bored, grouted-in-place, and incorporate steel reinforcement which resists most of the load (FHWA, 1997; ASCE, 1997). However, depending on the soil conditions and the performance requirements and levels of construction expertise, the concept of a micropile may be broadened. For example, the thriving underpinning market in Finland is largely served by jacked or percussed steel pipes installed to rock head and either grouted in place or not (Lehtonen, 2001). The uniformity of the overburden conditions and the predictability of bedrock elevations permit such straightforward and repetitive methods to be used.

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However, when considering deep foundation construction in karstic terrain, the highly variable and locally unpredictable nature of the ground invariably poses major problems. As designers become more aware of the advantages of the drilled and grouted micropiles as typically used in North America, the use of such micropiles to solve deep foundation problems in karst is becoming very common.

The purpose of this paper is to present an appreciation of the impact of karstic terrain on the design, construction and performance of micropiles. Details from recent case histories are provided to illustrate the special measures which may have to be taken, and the pile performance data which may be anticipated.

Aspects of Karst Geology

The term "karst" refers to a type of hummocky terrain found in the Karst area of the former republic of Yugoslavia. The area is underlain by carbonate bedrock that is prone to chemical weathering and solutioning and is characterized by sinkholes, caves, enclosed depressions, pinnacled bedrock, and other solution features. The term has been generalized to describe any area that has similar features. These areas are mainly underlain by carbonate bedrock such as limestone, dolomite, and marble but may also have gypsum or other types of rocks that are prone to the same types of solution or weathering features.

Karstic solution features are formed when water moves through interconnected openings in the rocks. Potentially weak or exploitable zones in the rock, such as bedding planes, joints, fractures, and faults, where they are enlarged by the solvent action of slightly acidic water. The solutioning process generally creates pinnacles, cutters, floating boulders, and caverns within otherwise competent hard rock strata. Overburden soils above such cavities can collapse into the voids forming sinkholes. (Figure 1)

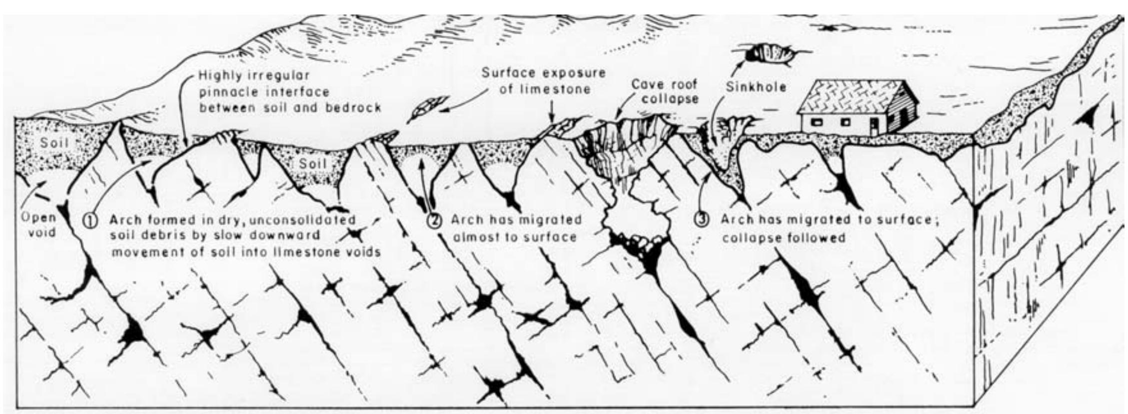

Figure 1. Progressive Development of Karst (Geyer, and Wilshusen,, 1982)

Downward migration, or subsurface erosion of soil into underlying openings in bedrock, can also occur as well as collapse of resulting cavities from construction equipment loads and related vibrations, or new structural loads. If there is a substantial decline in the water table, it may result in a loss of buoyant support and an increase in the groundwater flow velocity. These can also induce sinkholes by promoting void collapse and movement of soil into solution cavities.

Karst comes in many forms and is found in many areas of the United States and around the world. In areas such as eastern Pennsylvania, the limestone and dolomites of the Ridge and Valley and Piedmont, are of Ordovician and Cambrian age. These rocks have undergone extensive folding and faulting resulting in steeply dipping bedding planes and highly fractured rock. Marbles are encountered in northern Delaware. However, in areas such as western Pennsylvania and Ohio, the Pennsylvanian age carbonate rocks are more flat lying and have undergone only minor geologic stresses. The carbonate materials of Florida are somewhat different again, including softer and flat lying strata including, coquina, coral deposits and un-metamorphosed limestone and dolomite. The unconfined compressive strengths of these materials can vary from greater than 20,000 psi for the Ordovician limestone and Cambrian age marbles to less than 2000 psi for the oolitic limestone of Florida, and even lower for the gypsums. In general, some form of near surface carbonate material has been reported to be present in about 15% of the United States. (Figure 2)

Completing a representative study of a site underlain by karst can be difficult and expensive. An exploration consisting of borings conventionally spaced at about 100 ft centers is very likely to miss many karst features and may not extend deep enough to detect potential problems in the rock mass. A thorough study of these sites is warranted and should begin with research of regional conditions through the use of air photos, fracture trace studies, geologic mapping, remote sensing, historic mapping, local knowledge and site reconnaissance. This will provide a basis to develop a more site specific study which will be more likely to identify the true conditions.

Before test borings are drilled, additional indirect measurements can be made. Geophysical tools provide a means for indirectly measuring the potential variations across a site and identifying anomalous areas. Historically, tools such as ground penetrating radar, microgravity, shear wave, seismic reflection, seismic refraction, electromagnetic and spontaneous potential surveys have been used with varying degrees of success. Recent developments in computerized data collection, instrument sensitivity and post-processing have facilitated the use of methods such as Resistivity and Induced Polarization profiling which has proven to be very effective when performed, modeled and interpreted by competent and experienced personnel. (eg Bruce et al., 2001)(Dunscomb, 1999)

With an understanding of the regional geology and the potential site specific variability, the proposed development of a site can be considered, including size, earthwork requirements and foundation loads, and stormwater management issues. The invasive site exploration can then include traditional test borings, insitu testing and auger probes. Rock coring should also be performed in most cases to characterize the potential variability of the rock, to confirm that the apparent surface of the rock is more than a bolder or ledge, and to look for indications of voids. Additional exploration using air percussion drilling techniques often provides cost-effective means of supplementing traditional borings and gaining better site resolution of rock head variability.

The irregular conditions clearly can have a significant impact on construction in karst areas. Structures ranging from residential homes to the Petronas Tower in Malaysia have been constructed over karst. Figure 2 shows the location of metropolitan areas relative to underlying karst conditions in North America. As the infrastructure in these areas is repaired and expanded, treatment of the underlying conditions will pose significant difficulties. For example, several recent transportation projects in eastern Pennsylvania have incurred millions of dollars in ground treatment costs associated with karst conditions encountered during construction.

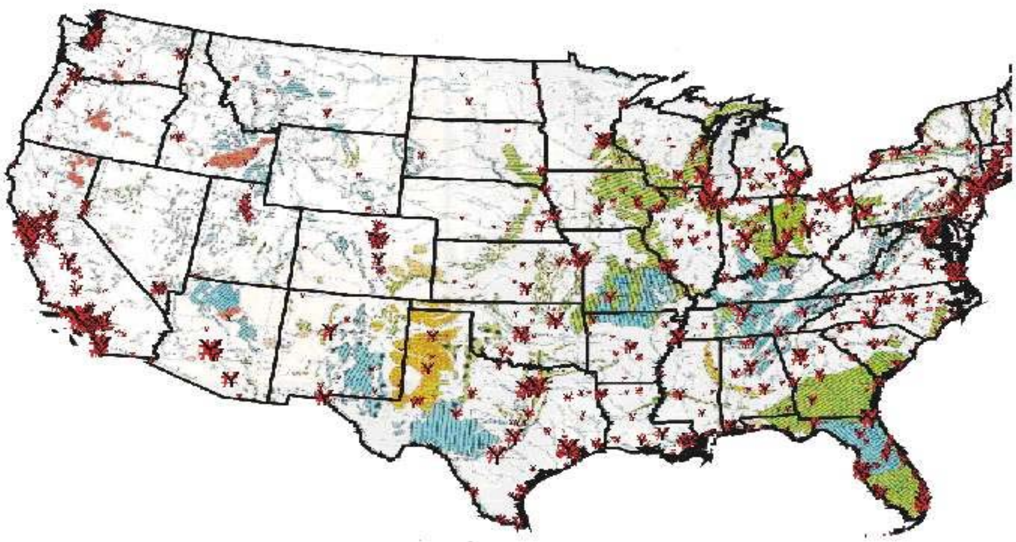

Figure 2. Engineering Aspects of karst, National Atlas, (WE Davies et al., USGS 1984)

Application of Micropiles

Micropiles in karst are used to provide structural support mainly as foundations for new structures, as underpinning of existing structures, and less frequently as part of a seismic retrofit where potentially liquefiable soils overly the bedrock. Regarding the underpinning of existing structures, micropiles are used to repair or replace existing foundations, to arrest or prevent movements, or simply to upgrade the existing foundation capacity. They are typically installed to resist axial loads, usually compression, although seismic applications will also involve lateral and/or tensile loading. Micropiles in karst are invariably designed as Case 1 (direct load carrying) and are typically Type A elements (FHWA, 1997), (Figure 3)

Their methods of construction afford a flexibility that is ideally suited to combat the rapid variations in rock head elevation, and within the rock mass, that characterize karst. Such methods also facilitate optimization of the overburden drilling process. Micropiles are therefore very adaptive to karstic conditions, and are amenable to providing detailed ground condition records, during the construction of every pile. Since they are typically designed to transfer load by skin friction (as opposed to end bearing), such construction observations can be used to ensure that the design requirements for bond length can be satisfied in each pile. In this regard, the existence of a major karstic feature just under the pile tip should not adversely affect micropile performance, as it would have the potential to do in the case of a large-diameter end bearing pile.

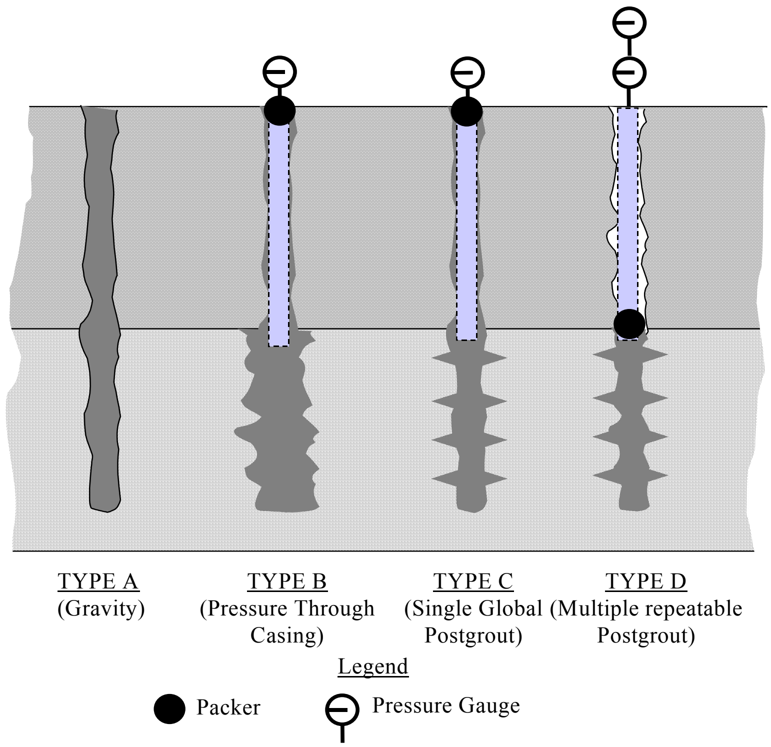

Figure 3. Classification of Micropile Type Based on Grouting (FHWA, 1997)

Design Issues for Micropiles in Karst

Since the material types used to provide pile capacity in karst can range from relatively poor quality rock and weathered materials that behave as sands and gravels to very hard and massive rock, the available grouting methods can be any of those shown in Figure 3, although the most common applications in the United States involve Type A. As described in the Case Histories section, however, both Type B and Type D have also been used.

Several key factors have to be considered when applying micropile technology in karst. Of prime concern is how the load to be carried by the rock, given that the most troubling issue with karstic rock is its inconsistency. Where the bearing materials are similar to granular sands and gravels such as the oolitic limestone deposits found in Florida, this can be a relatively straight forward decision. Applying good field exploration, observation, knowledge of the geomorphology of the area and past experience, the bond zone location and condition of a pile can be determined. However, if the conditions consist of highly variable soil conditions over very hard rock, the rock is often the most desirable material to develop significant capacities. These rock masses are often highly fractured and can contain voids, seams and caverns. The average, ultimate bond values of Type A piles in this massive hard rock have been shown to be in excess of 250 psi (Cadden, 2000). Thus, as with anchors in rock, bond lengths greater than about 3m rarely produce much increase in capacity. However, the elevation of the desirable rock for bonding is usually highly variable at a given site.

Several alternatives have been applied to accommodate these conditions. The most direct solution would involve drilling until 3 m of essentially massive rock is encountered. A more efficient pile would seek to consider the benefit of the discontinuous rock in some form. This is where the experience, sound engineering, and understanding of the local geology are critical. With such knowledge, the consideration of discontinuous masses of competent rock can be additive to reach the desired 3 m length. Great care must be applied to establish the suitable definition of a segment of competent, acceptable rock. This definition will include the minimum allowable length of each segment, the permissible length of discontinuity between each segment, and the acceptable material within this discontinuity.

When establishing these guidelines for each site, consideration should be given to the potential for an apparent rock mass to be merely a floating boulder (discontinuous from the main rock mass), for the possibility that a rock zone is a ledge or pinnacle that may break under load, and the impact on significant pile length difference within a pile cap or between columns. Table 1 provides a sample of how such a criterion may be developed for a site to manage potential variations in conditions.

Table 1. Conceptual management strategy of variations in site conditions.

	Example 1	Example 2	Example 3
Required Total Length	3 m	3 m	3 m
Minimum Rock Segment Length	3 m	1.5m	1m
Allowable Discontinuity- soil filled or open	-	0.5	0.3m
Allowable Discontinuity- broken rock filled	-	1m	1m
Maximum Total Allowable Discontinuity length	-	1	2

Assumptions: Bond zone is grout tight
No known voids beneath pile tip
Bond rock not suspected to be floating boulder or unstable ledge/pinnacle

The type of reinforcing used for the pile is variable and is often limited by the drilling methods utilized. Both pipe and central bars are common types of reinforcement. Each of these can be used exclusively or the two in combination. With all micropiles, the method of installation must ensure that the reinforcement throughout the bond zone is adequately embedded in grout. This can be achieved in a stable hole through tremie grouting. However, in more difficult conditions, care must be taken to ensure that the grout fills the bond zone completely and remains in place as the reinforcement is placed, and before setting.

Grouting of the pile is most often achieved with a cement based grout composed of water and Type I or III Portland cement having a water cement ratio (w:c) of about 0.45 to 0.5 by weight. Often, admixtures such as fluidizers and retarders are utilized to facilitate pumping. Accelerators or clays are not recommended due to potential corrosion and loss of friction associated with these materials. These grouts have little difficulty achieving compressive strengths of over 4000 psi. Often the inconsistencies in karst geology will result in grout loss from the drill hole. Therefore, for economic reasons, alternative grouts including sand and other fillers can be used to fill voids and stabilize the pile hole. The filler grout is generally not suitable for load transfer and the bond zone must be thoroughly cleaned after drilling and before final grouting is performed.

Where very soft soils or voids are penetrated, or where there is a future potential possibility for a sinkhole to develop adjacent to a pile, structural buckling of the pile may be a consideration. Questions also arise about buckling potential where the hole drilled is slightly larger than the casing used for reinforcement of the pile and the grout coverage in the annulus space is not assured. As presented in Bjerrum (1957), only very limited strength within the soil is required to result in the pile being supported continuously along its length. In materials such as peat, unconsolidated soft clays and loose silt where the elastic moduli is less than about 70 psi, buckling should be evaluated. Thus, nearly any soil present around the pile, including material that will squeeze to fill the annulus space is considered adequate for lateral support. Only where there is reason to believe that the pile will penetrate a void which is not filled with grout during construction, is it necessary to design for special buckling considerations. For the long-term condition where future sinkholes are likely, redundancy in the pile system or other advanced ground treatment must be considered for the site.

Influence of one pile on another is generally not considered to be an issue where the center to center spacing exceeds three times the pile diameter. This has been shown to be reasonable for micropiles as well. However, given the extremely high capacities that can be achieved with micropiles in rock, the group effect and the overall rock mass strength must be considered.

Given the highly variable conditions typical of karstic bedrock, the lengths of the micropiles above the bond zone may vary significantly. Often these lengths can be through overburden soils which will provide little if any resistance to the vertical loads. As such, elastic movement of adjacent or nearby piles above the bond zones can vary dramatically in service. Load testing of adjacent piles at the Exton Mall Project in

Pennsylvania (Cadden et al., 2000) depicted this impact: tests of two adjacent piles, which varied in lengths from 21.5 to 30.5 m, resulted in nearly 10 mm in differential movement of the piles at design load and 25 mm at test load. This difference in stiffness in a pile cap will result in redistribution of loads to the pile and rotation of the cap element. Care should be exercised to evaluate these conditions when encountered in a foundation element or to design the entire foundation system to account for such anomalies by tying foundation elements together to limit potential rotation. Alternatively, stiffening of longer piles through the placement of additional steel in the element can be used to create more uniform movement performance.

Construction

Micropiles are constructed in essentially three steps: (a) drill hole to pile tip elevation, (b) install reinforcement, and (c) install grout. Karstic conditions offer challenges to all three steps, motivating micropile contractors to use the most efficient combination of procedures to optimize installation. For instance, using the drill casing or rod as the steel reinforcement eliminates one of three steps and may reduce labor cost, but increases material cost. Does the faster production rate offset the increase in other costs sufficient to use the method? Can the thread design of the drill string or casing withstand the loads to be placed on the pile reinforcement? While each site is unique, and in fact, each hole can be unique and essentially a new job, certain methods have proven effective in recent years. These commonalties are now reviewed.

Drilling From an engineering viewpoint, the drilling method selected for a particular project should be dictated by the rock and overburden conditions at site. However, drilling techniques for rock and overburden are commonly driven by the type of equipment and hardware or "tooling" available to the contractor. Since the equipment and the tooling are very expensive, contractors are reluctant to change tooling systems from job to job. Consequently, a contractor will prefer a particular method over others simply because it is in the contractor's yard and within his field of experience.

Drilling overburden is accomplished through the use of six methods and drilling rock or weathered rock by three (FHWA, 1997) (Figure 4). Drilling in karst ensures that each pile location will encounter overburden, rock, and weathered rock and the conditions will be encountered at highly variable elevations, with the potential for voids at great depth. All of the drilling methods, with the exception of "Single Tube Advancement" (Method 1) are effected by the ability of the driller to maintain an open path for drill cuttings to escape the bore hole. If the cuttings are allowed to drop on top of the bit assembly at the cutting face, the drill string may become locked in the hole, or at the very least, drill production will be dramatically reduced. Since karst is characterized by fractured rock, weathered seams and voids, there is great potential to lose the air and cuttings prior to reaching the top of the hole. Therefore, the most effective drill tooling systems are "Duplex" drilling systems, i.e. those that advance the casing down hole with the drill string as depicted in Figure 4 (Methods 2, 3, 4 and 5), thus protecting the hole stability by mechanical means.

In addition, micropiles require the use of steel reinforcement to carry the load, and a cased hole guarantees that the reinforcement may be successfully inserted to the target depth..

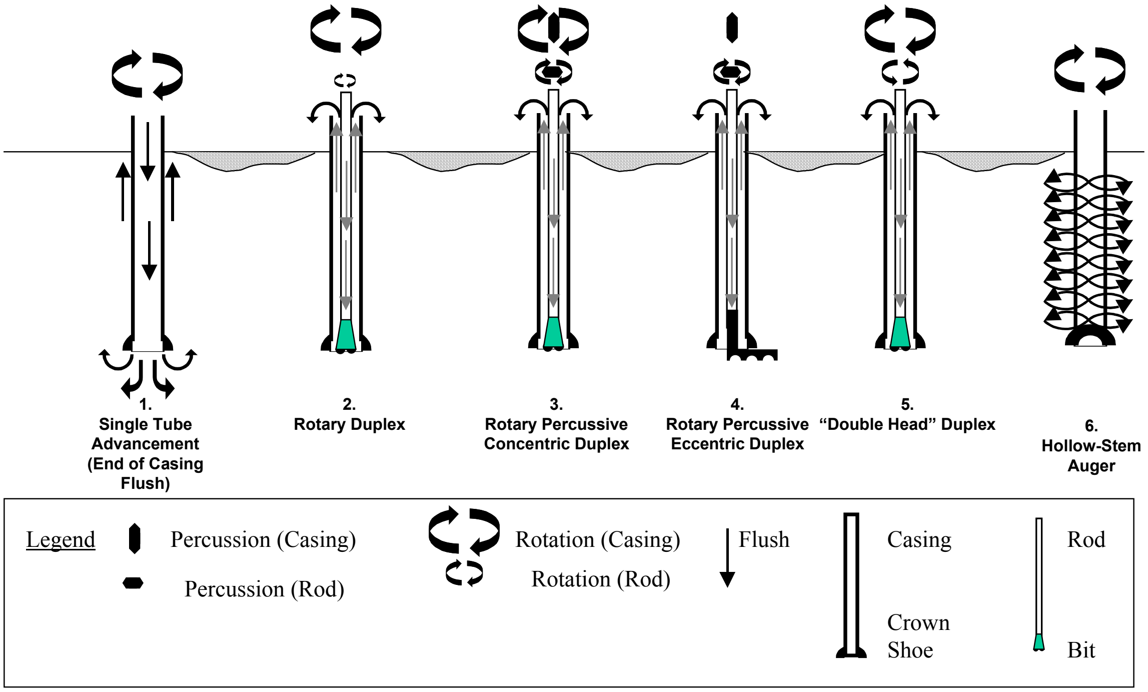

Figure 4. Schematic representation of the six generic overburden drilling methods (FHWA, 1997)

Typically, oil field casing of 5.5", 7" or 9 5/8" o.d with a wall thickness of 0.5" are readily available and can be duplexed down hole with a casing under-reamer, concentric bit or eccentric bit.

Various proprietary systems exist and may use casing under-reamers, knock-off ring bits, or eccentric bits to over cut the hole, allowing room for the casing to follow the rock bit into rock. Systems such as TUBEX and CENTREX are readily available, and may be run on modern overburden drilling rigs. For the most demanding situations, such as steeply bedded, highly fractured karst with voids, "Double-Head Duplex" systems, as shown in Figure 4 (Method 5) with counter-rotating inner and outer drill strings are highest in cost, yet most effective with the highest penetration rates and a "guaranteed hole" capability in most circumstances.

Open hole techniques are used where a stable, free-standing hole is geologically possible. Their use is often precluded in karst for micropiles, because an uncontaminated bond zone is required for the transfer of load from the grout to the rock. Open hole techniques may be used where the depth of holes is shallow (less than 30'). The risk with an open hole technique is that the removal of the drill string allows overburden, fractured rock, sands below the water table and other materials to fall down into the bore hole and contaminate the bond zone, or cause the embeddment to be too short. Therefore, the hole may need to be re-drilled, and in many cases, a casing advancement system such as those described above is ultimately needed for the difficult areas of the site. The potential for hole collapse, resulting in a drill string with an expensive down-hole-hammer stuck 100 feet in the ground implies that the drill rig selected should have sufficient torque and retract force to extract the string. Water injection seems to aid initial hole stability and lubrication, thus reducing the potential for a lost drill string.

Single tube advancement using air or water flush is not recommended because the flushing medium may further unravel the karst structure by flushing clay from seams, or inflating the overburden. Continuous-flight auger sections (Method 6), as well as percussive single tube advance (Method 1)with a lost bit are not recommended and cannot drill rock efficiently.

Drilling in karst may produce large amounts of the clayey residual weathered carbonate limestone or dolomite. This red- brown clay will be wet and quite slick and if left unmanaged, will render a site impassable. Spoils removal is necessary for the management of a clean site. When drilling indoors, duplex drilling systems are advantageous because a spoils collar may be installed on the rig, capturing the spoils and diverting them out of the structure through a hose.

As noted above, micropile design in karst requires an awareness of relative pile lengths within a pile cap. The drilling sequence of piles must also be carefully planned so that ungrouted or uncured grouted pile bond zones are not compromised during the drilling of adjacent or nearby piles. In general, a distance of 15 feet is sufficient to reduce the risk of interconnection of drilling air between pile locations. However, in karst conditions, drilling air may travel great distances due to open karst features. On karst sites, careful observation of drilling processes on a hole by hole basis is the best quality control method. When drilling through or near existing foundation elements, the flushing air or water may undermine the footing or pile cap. In these situations, duplex drilling, again, provides the best risk management, as most air and cuttings are diverted to the surface through the internal flush between the inner and outer drill string. A simple method that may prove efficient in this case is to rotate a disposable casing to suitable depth, allowing subsequent drill cuttings to return directly to the surface.

Grouting As noted above, the grouting process defines the type, and often the capacity of the micropile. The grouting competency of the contractor may, above all else, be the most critical variable in the performance of a micropile. The reinforcement steel, hole dimension, grout strength and pile design can be measured quantitatively, but grouting is a process, and a process can only be evaluated subjectively.

Micropile grouting equipment consists at a minimum of a colloidal high speed, high shear mixer, holding tank with agitation, grout pump capable of reaching pressures of 300 psi, pressure gauges, recirculation lines, qa/qc equipment and log books. The colloidal mixer is a high-shear grout plant that is capable of rapidly mixing neat cement based grout in a few minutes, with a thorough wetting of the individual cement grains. A thorough wetting allows a low water-cement ratio grout to be pumped easily through the grout lines that run from the plant to the pile. Without a colloidal plant, clumps of cement will cling together, clogging injection lines, and ultimately yielding a lower strength grout, because significant amounts of the cement grains are not hydrated. Following a thorough mixing, the grout must be stored in an agitation tank with agitation blades that constantly stir the mixed grout, prolonging separation of the cement from the mix water. With proper admixtures, grout life may be extended easily to a working time of 6 hours, and in some cases, may be suspended indefinitely until the reaction is re-initiated on demand. A Marsh Cone simplifies optimization of the grout mix design with respect to retarders and fluidifiers. A Marsh Cone time of 11 – 14 seconds is typically considered ideal for pumping into micropile packer systems.

Grout strength is typically required to be measured at 7 and 28 days, and to reach over 4000 psi. Since the grout for micropiles is essentially neat cement and water, the specific gravity, measured at the time of mixing and injection, can be extrapolated to the strength at cure. For instance, a grout mix with a water cement ratio of 0.45, with an ultimate strength at 7 days of 4000 psi, will have a specific gravity of approximately 1.89. If the density of the grout is less than this figure, the batch may be discarded, and a new batch mixed for testing and injection.

Grout pumps are typically progressive cavity style pumps, such as Moyno® pumps because the flow rate and pressures remain nearly constant during recirculation and injection phases. Grout injection pressures at or below 100 psi for Type B piles is typical, yet Type D piles may require injection pressures peaking at over 1000 psi for re-grouting through initially set primary grout. This will require piston or ram pumps (Muller and Bruce, 2000) Recirculation lines and injection manifolds allow the grout to remain in motion during non-injection sequences, preserving the fluid characteristics and stability of the grout. If the grout is allowed to stand still in the hoses running from the plant to the pile for an extended period of time, the grout will stiffen and begin to approach an initial set. Re-agitation of the grout will cause a drop in final strengths at cure.

Installation Records The micropile contractor must keep accurate and contemporaneous logs showing detailed installation information. Boring logs are used for planning the pile installation, but in karst conditions, rock head may vary greatly within a few feet of the boring. In karst, it is concievable that each pile will be unique in overall length, un-bonded length, depth to rock head and sequencing of adequate bearing stratum, as shown in Table 2. The pile installation log is the only record of the verification of the pile design dimensions and compositions having been achieved. The pile installation log is also generates the pay items for the project, such as drilled footage in soil as well as rock, and amount of grout consumed. The contractor must be able to verify through his logs that the design requirement of a given amount of rock has been encountered in order to substantiate that a pile of the design capacity has been installed.

Illustrative Case histories

Warren County, NJ (Bruce, 1989)

<u>Background</u>: conventional H piles could not be installed as foundations for a bridge pier near the Delaware River. A total of 24 micropiles each of 100 tons working load were designed, which proved more economic than an alternate based on six large diameter end

bearing (36-inch) caissons . Also a load test was foreseen, which would give the Owner comfort that the individual micropile capacity could be attained (in skin friction).

Site and Ground Conditions: the bedrock was a Cambro-Ordovician dolomitic 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 were intersected even over 100 feet below the surface, for example, 50 feet of soft brown silty clay below 106 feet at the location of pile 24. Dipping 55° to the southeast, the rock mass proved highly variable laterally and vertically.

Geotechnical Issues: design of the bond length was based on a maximum average rock/grout bond at working load of 50 psi, and so a bond length of 8½-inch-diameter and 15' feet long in competent rock was chosen for these Type 1A piles. Recognizing that the rock was likely to be very variable, provision was made to allow the 15-foot bond zone to not necessarily be continuous. In most piles this was subject to the following restrictions:

- The lower part of the zone was to contain at least 10 feet of continuous sound rock.
- Soft interbeds were to be less than 3 feet thick.
- A zone of acceptable load-bearing rock was to be at least 5 feet thick.
- Regrouting and redrilling of interbeds within the overall bond zone was to be undertaken if required.

However, certain piles were judged especially critical structurally, (mainly on the footing perimeter) and were required to have a 15 foot continuous bond zone.

Construction Issues: individual pile lengths varied from 44 to 200 feet across the 21-foot x 17-foot site. Compared to the allowable deviation of 2°, no pile exceeded 78% of this figure, while the overall average was 0.8°. Throughout, the very adverse conditions posed serious drilling problems, resolved only by repeated pregrouting and redrilling (At the time of construction, no experience existed in the use of a percussive duplex method, such as Tubex, to stabilize such conditions). Poor or voided rock was consistently found below the anticipated elevation of the alternative caissons, further justifying the use of micropiles. Overall, the total drilled length of 1920 linear feet corresponded with the total foreseen quantity of 1710 linear feet. Variations from 43 feet less to 30 feet more with respect to foreseen were recorded on individual piles, highlighting the variability of the rock. Overall, a volume of grout equivalent to four times the nominal hole volume drilled was injected. Much of this was consumed during pregrouting operations. The level of maximum takes corresponded with groundwater level.

Performance/Testing: a special test pile, 30 feet long with a 5.33-foot-long bond zone was subjected to a static compression load test to just over twice working load. Procedures were taken to isolate the bond zone and prevent end bearing. At Test Load, the Elastic Ratio (Bruce et.al., 1993) analysis configured that only the upper 25 feet of the pile was acting as if debonded, i.e., that practically no debonding had occurred in the rock socket, even at nominal average grout/rock and grout/steel bonds of 304 and 250 psi respectively.

Creep at Test Load was 0.011 inch in 60 minutes. Testing to failure of the steel pipe reinforcement was then undertaken (224 tons) without apparent geotechnical failure occurring.

Overall, the bridge has been completed since the late 1980s and the performance of the micropile supported pier has proved totally satisfactory.

San Juan, PR (Zelenko et al., 1998)

Background: the U.S. Post Office and Courthouse building in Old San Juan was built between 1914 and 1940 on driven steel and timber piles. Concerns about the potential for liquefaction of the underlying sands led to a major seismic retrofit, which included the use of 217 micropiles, each with a working load of 60 tons compression, 40 tons tension, and 5 tons in lateral capacity at a maximum allowable deflection of 0.5 inch.

Site and Ground Conditions: underlying 8 to 10 feet of variable fills and about 25 feet of fine to coarse sands, was the weathered limestone bedrock. This stratum had a variable rock head elevation (varying by about 15 feet across the site), and a very variable consistency (N = 11 to > 100), ranging from soft limestone to residual silts and clays. The micropiles were all located inside the building, where access was often severely limited and work was conducted in low headroom conditions. During construction, a number of specific challenges were faced, including:

- Simultaneous overhead demolition and site clearance.
- Presence of an old basalt fortification wall dating from the 1600s.
- Presence of numerous other drilling obstructions, such as existing piles, pile caps, masonry footings, and remains of prior structures (none accurately recorded in available drawings).
- Water rationing and high ambient air temperatures.

These challenges were overcome by exploiting the flexibility afforded by judicious micropile design, and the quick responsiveness of the construction methodologies.

Geotechnical Issues: the drill casing (9½-inch o.d.) had to be embedded at least 3 feet into the limestone, but not less than 25 feet below the existing floor slab. It was also required to extend the bond zone a further 14 feet into the rock, and to have a minimum diameter of 8 inches.

Construction Issues: in addition to the challenges listed above, the instability of the bond zone after drilling, plus the high ambient temperatures, led to each hole being initially tremie grouted through the drill rods, prior to their removal, with a retarded, fluid neat cement grout, colloidally mixed. Following insertion of the core steel reinforcement, the bond zone was then pressure grouted to promote enhanced bond capacity in the weak, weathered limestone (Type 1B pile).

Performance/Testing: four load tests were conducted to various acceptance criteria. The first pile installed was selected for compression testing and had experienced construction problems resulting in a bond zone only about 11 feet long. The first loading of 200%DL (120 tons) provided a permanent movement of 2.056 inches which was not acceptable.

This pile was then subjected to regrouting (i.e., became a Type 1D pile), and was retested (Figure 5), illustrating the action of postgrouting in minimizing permanent movement. A further pile was tested in compression using revised construction techniques and performed acceptably (Type 1B). Lateral, and tensile testing was also conducted on other piles, and each met the project acceptability criteria. The tensile and compressive testing performances are shown in Figure 6 for comparably sized piles.

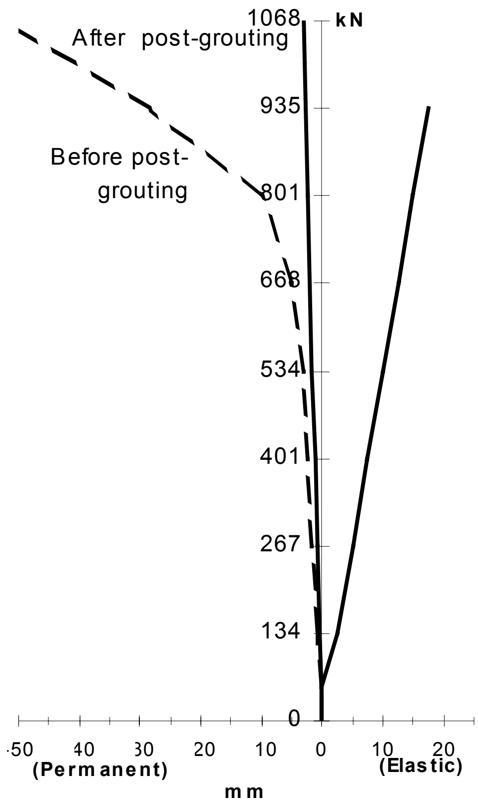

Figure 5. Load vs. Movement (Zalenko, et al. 1998)

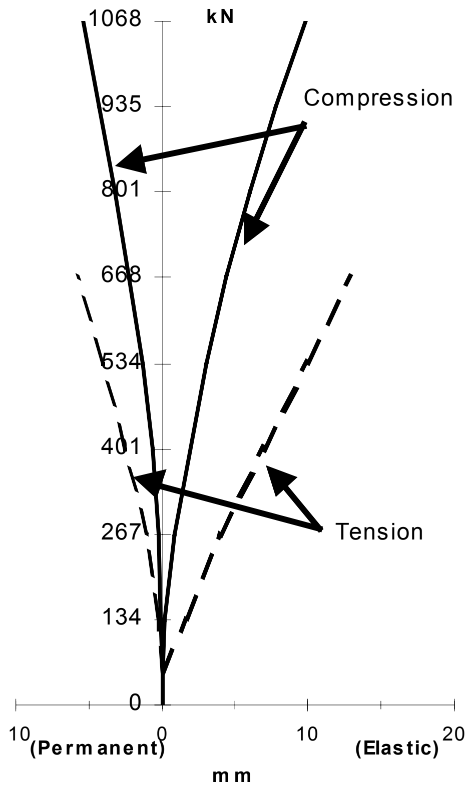

Figure 6. Load vs. Movement (Zalenko, et al.1998)

Exton Square Mall, PA (Cadden et al., 2001)

Background: A major expansion of an existing and fully operating shopping mall, including two parking garages, had to be conducted over a karstic dolomitic limestone bedrock. Micropiles were chosen for the deep foundations since they were judged best suited to the tight access, load headroom, logistical restrictions, highly variable rock head, irregular rock mass qualities, and locally high individual pile working loads (300 kips). An intensive preproduction load test program was undertaken. The existing structure required 405 piles (294 interior) and the garages needed 355 piles.

Site and Ground Conditions: The limestones regionally are of the Chester Valley Sequence of the Piedmont Physiographic Province. Locally, they are represented by the Ledger Formation, a medium to coarsely cystalline dolomite containing oolitic, siliceous and cherty beds. The susceptibility of the site to karstic solutioning was reflected in the mapping of three sinkholes on the site, and the occurrence of subsurface cavities and pinnacles locally. Furthermore, a limestone quarry had previously been located on the west side of the site, and was now backfilled to a depth of 27 feet. Typically overburden consisted of alluvial deposits (6 to 14 feet) overlying residual clayey soils to depths of up to 81 feet.

Geotechnical Issues: The bond zone was required to comprise 10 feet of competent rock and a diameter of 7½ to 8½ inches. Even the better quality rock was noted to have significant fissuring, giving the potential for grout loss from the bond zone.

Construction Issues: Both Type 1A and 1D micropiles were experimented with during the construction optimization phase. The piles for the existing structure varied from 20 to 150 feet (average 34 feet) below existing slab, while the piles for the garages varied from 25 to 180 feet (average 43 feet). The conceptual advantage of the Type 1D pile (wherein regrouting sleeves were installed on the full length 7-inch-diameter core reinforcing pipe) was that pregrouting and redrilling could be eliminated and the integrity of the grout around the bond zone could be verified by postgrouting. During drilling of some test piles, running sand conditions were encountered below the water table which appeared to pack around the sleeved reinforcement pipe so preventing routine postgrouting being accomplished. Such difficulties, rendered the simple Type 1A approach with (often, repeated) pregrouting and redrilling the more practical and cost effective option on this particular site, even though load testing confirmed the ability of the Type 1D to satisfy design requirements.

Performance/Testing: Test Piles 1 and 2 were rejected due to construction problems or testing equipment failure. Tests 3 through 5 featured Type 1D piles, and Tests 6 through 10 were performed on Type 1A piles. Data are shown in Figure 7. In general, excess movement was ascribed to incomplete grouting of the distal 2 feet of the bond zone (as a result of collapsing soil). Data on Test Piles 3 through 10, shown in Table 2, confirmed that properly installed micropiles of either type would successfully carry loads of up to 420 tons with total movements less than 2 inches. Based on these tests, average rock-grout bonds of 200 to 300 psi (without failure) could be inferred for this rock.

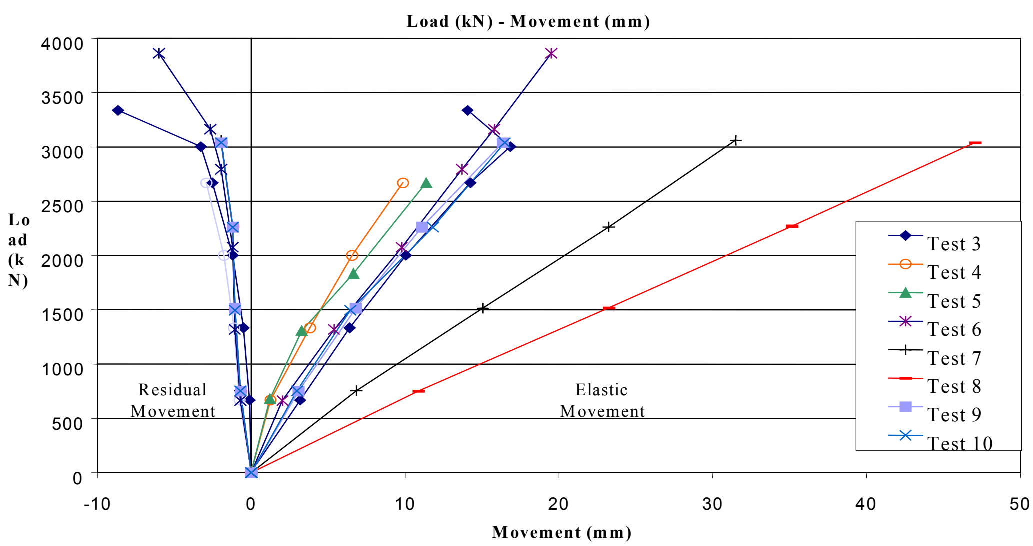

Figure 7. Load vs. Movement (Cadden, et al. 2001)

Load Test	Total Casing Length (meters)	Bond Zone (meters)	Grout Zone Diameter (mm)	Max. Load (kN)	Max. Total Deflection (cm)
1	6.1	None	218	667	—
2	6.1	3	218	2,669	11.4
3	10.6	3	218	3,336	2.3
4	8.8	3	218	2,669	1.3
5	10.4	4.6	218	2,669	1.3
6	11.9	3	188	3,638	2.3
7	30.5	3.2	188	2,669	3.3
8	21.5	3.2	188	2,669	4.8
9	10.7	3	188	2,669	1.9
10	12.2	4.6	188	2,669	3.3

Table 2. Load Test Data - Exton Mall, PA ,Cadden (2001)

Conclusions

This paper illustrates that the natural variability of karst must be met with thoughtful and appropriate design methodologies, and in responsive, flexible construction techniques of high quality. A key first step towards achieving project goals in any ground condition is a proper evaluation of the site. This is particularly the case in karst where conditions may be expected to vary considerably over small lateral and vertical extents. With regard to micropile works, accurate rock-head contouring (isopachytes) and thorough investigation of the variations in the rock mass itself are especially critical.

Regarding construction, a wide range of drilling and grouting methods are used, characterized by the two extremes of sophistication. On the one hand, holes are initially drilled "open", knowing that some form of pre-grouting and re-drilling will be necessary to provide stable, grout tight conditions for the placement and bonding of reinforcement. On the other hand, relatively sophisticated duplex drilling methods can be used so that hole stability is assured by an outer casing (temporary or permanent), with grouting carefully conducted to ensure that the design requirements relating to the load transfer and corrosion protection are met. Grout takes in karst may be expected to greatly exceed the nominal hole volume. However, such takes cannot be predicted with any degree of accuracy in advance, and must be regarded – and paid for as a major project uncertainty.

Assuming that piles can be installed appropriately, data from several sources can be tabled to verify excellent performance under service or test loading conditions. Whereas postgrouting may be necessary to provide high bond values in very weathered limestone, gravity grouting will provide such high values in fresher limestones, even if they contain significant karstic features which will require pretreatment. Construction in karst is always problematic and the installation of micropiles in such conditions is usually no exception. However, micropiles have such a degree of design and construction responsiveness and flexibility that renders them a superior option in such conditions.

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