

THE CHALLENGES OF CONSTRUCTION IN KARSTIC LIMESTONE: THREE MAJOR CASE HISTORIES

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The construction of major structures in karstic limestone terrains poses difficult problems to designer and contractor alike. This reflects the inherent variability of such conditions, and also the significant impacts caused by the construction processes themselves. Three case histories are provided of recent projects in the United States: the sealing of a major inflow into a quarry, the improvement of a highly loaded bridge pier foundation, and the installation of high capacity micropiles. Emphasis is placed on problem assessment and execution verification.

1. INTRODUCTION

The construction of large civil engineering structures in karstic limestone regions always merits – and usually demands – special engineering attention. Problems may be associated with constructing in or on the rock mass itself, or with constructing on natural overburden or placed fill overlying the rock mass. In the latter case, the sudden and invariably surprising developments of sinkholes causes a threat not only to the structures themselves, but also to the personnel and equipment engaged in the construction processes.

To engineers not well versed in the physical and chemical geomorphological factors which create such terrains, the whole issue may appear to be confusing, illogical, and unpredictable. Typical questions are of the type “why are the drilling logs so different over such a small area”, and “why did that sinkhole just *suddenly* appear?” To engineers who chose to lend credibility to the concept of physical and chemical processes which act over hundreds of millions of years, yet which can be suddenly catalyzed to generate catastrophes in a matter of seconds, the conundrum can be rationalized. Thus we can understand – due to the creation of “pinnacles and valleys” in the limestone rockhead by deep chemical weathering of vertical joints – why the borehole drilled through a pinnacle will indicate solid limestone for its entire length, whereas the borehole drilled only 1 m away in plan will encounter nothing but plastic red clay extending for tens of meters below its neighbor’s “rockhead elevation.” Likewise, the puzzle of sinkhole “swarms” developing after major changes in regional piezometric levels resulting from, for example, dewatering in an adjacent deep limestone quarry, can also be solved. The lesson from the first problem is that one of the few predictable things about karstic limestone terrains is their unpredictability. The lesson from the second problem is that human interference with structurally metastable geotechnical situations can lead to distinctly rapid responses from the laws of both nature and gravity.

As specialists involved in the analysis, design, and construction of deep foundation systems in such terrains, the authors have developed a strong appreciation for the variability of karst, and for the potential impacts that construction processes can exert

on it. The problems the authors have encountered in their practices in the Eastern and Central United States are not, of course, generically different from those their colleagues in karstic terrains elsewhere in the world continue to face. The purpose of this paper is simply to share the nature of their response to three commonly encountered problems:

- The sealing of massive water flows through karstic features.
- The construction of a massive bridge pier on a karstic foundation.
- The installation of micropiles in a karstic terrain.

Each case history is presented in standard format, namely

- Project and problem evaluation.
- Design of solution.
- Execution of solution.
- Control and verification of solution.

2. SEALING A MASSIVE INFLOW

(Major reference: Bruce, Traylor and Lolcama, 2001)

2.1 Project and Problem Evaluation

A very large and strategically important limestone quarry in West Virginia suddenly developed a basal inflow of over 133 m³/min following routine blasting and a period of abnormally severe rainfalls. Prior to the incident, total inflows from all sources had been substantially less than 38 m³/min.

A team of hydrologists, geophysicists, geologists, and geotechnical engineers convened, and combined to identify the source and path of the inflow. This was determined to be along a karstic feature extending from an eddy in the Shenandoah River (El. 90 m) and intersecting a thrust fault daylighting in the quarry (Figure 1) – a distance of around 800 m. Attempts to seal the leak at the source (with sandbags) failed, and the quarry was allowed to partially and temporarily flood to El. 79 m, from its floor of about El. 39 m. Even at this reduced hydraulic gradient, the total inflow was estimated at over 83 m³/min, passing through a major karstic system in the Ordovician dolomites and

limestones. The individual features were as large as 15 m in maximum dimension, and were believed to be as deep as 60 m. They were most prevalent in a zone about 150 m wide ("White Line" – Figure 1).

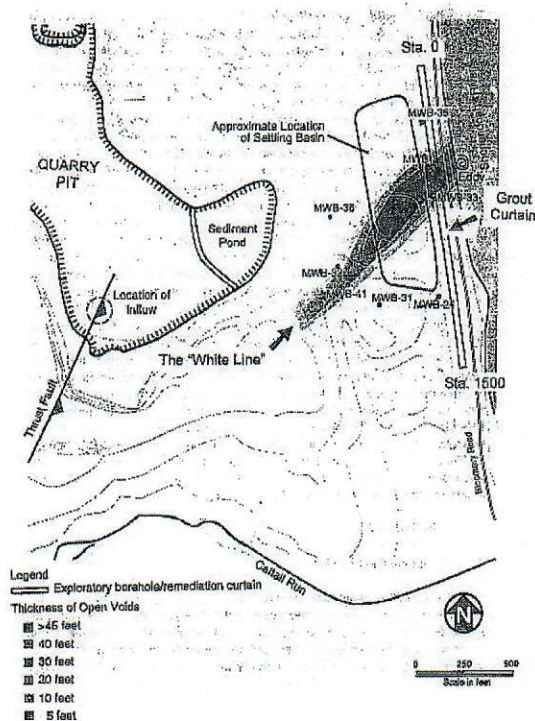


Figure 1. Quarry in West Virginia (1 ft $\equiv$ 0.3048 m).

2.2 Design of Solution

The goal of the solution, set by the Owner, was to reduce total inflow into the quarry to 30 m³/min from all sources. Hydrological studies indicated this meant reducing the river inflow to about 11 m³/min. Various options were, understandably, reviewed, including the construction of a large diameter overlapping pile wall (e.g., Bruce and Dugnani, 1996). Based on financial, logistic, schedule, and technical reasons, a grout curtain was selected, intended to both treat "latent" karsts and penetrate fissured rock in a preemptive fashion, and to create a permanent plug against the water flow in the "active" karst as the responsive solution. This curtain was to extend (Figure 1) for about 500 m laterally and 80 m vertically.

The main challenges to the success of the operation were the high water flow velocities and rates, possibly through multiple conduits; the presence of mud-filled karsts, therefore potentially susceptible to subsequent "blow out"; and the possibility of unconfined grout migration into the river. The following decisions were made:

- The MPSP system (Bruce and Gallavresi, 1988) would be used to inject high mobility cement based grouts to seal fissures in the less solutioned bedrock around the karstic features.

- End-of-casing grouting techniques would be used to inject low mobility grouts (Byle, 1997; Cadden et al., 2000) to seal karstic features with low seepage flow velocities.
- Hot bitumen (over 200°C) would be injected via special delivery lines and pipes into the regions of fastest flow (Bruce et al., 1998), simultaneous with the adjacent injection of additional high mobility grout as a "brittle reinforcement" and to compensate for natural shrinkage of the bitumen upon cooling.
- The curtain initially consisted of two rows of vertical holes, 3 m apart, the holes in each being at 6-m centers. Several holes were added during later phases of treatment to facilitate "tightening" of the curtain and allow intense treatment of specific zones.

2.3 Execution

Drilling was conducted by a variety of methods ranging from simple down-the-hole hammer to rotary percussive concentric duplex (Xanthakos et al., 1994). Temperature profiling of each hole was conducted to help establish the zones of strongest water flow (the river temperature was higher than that of the normal, static groundwater).

High- and low-mobility grouts (Tables 1 and 2) were injected with conventional equipment, and in accordance with standard U.S. operating processes. Specially prepared pumps and ancillary equipment were used to inject the bitumen at rates of up to 25 m³/hr. All materials and processes were subject to stringent QA/QC testing and standards.

By the conclusion of the project, which involved nine different phases of treatment to ensure full responsiveness to the ground conditions (and the interim changes triggered by the grouting operations themselves), totals of grout volumes injected were as follows:

High mobility grout (slurry)	6465 m ³ (in 173 holes)
Low mobility grout	1649 m ³ (in 147 holes)
Bitumen	4724 m ³ (in 58 holes)

Table 1. Low Mobility Grout (LMG) Mix Design, Quarry, West Virginia

INGREDIENT	QUANTITY
Water	Approx. 190 L
Cement	160 kg
Flyash	160 kg
Sand	950 kg
Antiwashout agent: UW450	3 L
Polypropylene Fiber	1.4 kg
Maximum Slump	$\leq$ 50 mm

Table 2. High Mobility (Slurry) Grout Mix Designations, Quarry, West Virginia

INGREDIENT	MIX TYPES				
	A0	A1	A2	A3	A4
Water (L)	310	401	390	397	386
Bentonite* (L)	257	76	98	98	95
Cement (kg)	331	389	399	406	393
Flyash (kg)	290	340	349	355	393
UW450 (ml)	562	651	651	680	651
Dispersant 2000B (ml)	1833	2159	2188	2247	2159
Marsh Time (sec)	55	80 - ∞	120 - ∞	∞	∞
Bleed %	0	5 - 6	2 - 5	2	2
Specific gravity	1.53	1.55	1.6	1.63	1.64

* As in 8% Bentonite Slurry (prehydrated 24 hours)

2.4 Control and Verification

Throughout the treatment, data were recorded and analyzed from various sources, in order to try to understand the impact of the operation and increase its effectiveness:

- All relevant drilling data and grouting parameters
- Piezometric data
- Temperature and pH variations in downstream standpipes
- Measurements of flow rates into the quarry and visual observations of the eddy in the river.

Many phases of treatment were required due to the propensity of the karstic system to "blow out" when subjected to an increased hydraulic gradient due to the attempted dewatering of the quarry. However, logical and determined efforts to achieve the project goals were ultimately rewarded by the achievement of a full and durable seal. At the conclusion of the work, the total flow into the quarry from all sources, was about 11.4 m³/min, at a differential head (relatively to the river) of 43 m. It was calculated that virtually all the flow from the river had been intercepted and stopped. Full quarry operation has resumed and the curtain has been fully effective since December, 1998.

3. JET GROUTING A BRIDGE PIER FOUNDATION (Major reference: Bruce, Hague and Hitt, 2001)

3.1 Project and Problem Evaluation

A major new 4-lane bridge is being built across the Mississippi River near Cape Girardeau, MO. Pier location considerations were an integral part of the preliminary studies, and were key to the recommendation to design a 3-span symmetrical cable-stayed structure. Being within 80 km of New Madrid, MO, the area is potentially very seismic. In addition, the rock mass underlying the riverbed sediments is

Ordovician limestone. The piers were all originally thought to be located on solid limestone, which permitted the use of conventional foundation systems such as spread footings or drilled shafts.

However, the footprint for Pier 3, the mid-river tower, was found during construction to be especially weathered, and indeed displayed major subvertical karstic features over its 33 x 20 m plan, to depths of over 15 m below average rockhead elevation (El. 76 m). No voids were found, and the features comprised red residual clay or contained deposited alluvial material. Some of the rock between the features also proved to be relatively soft and very weathered.

3.2 Design of Solution

Many alternative construction techniques were assessed before the decision was made to improve the rock mass by locating and treating the karstic materials so as to provide stiff in situ buttresses between the fresher rock "columns": this was driven more by seismic response concerns than static demands. Given the nature of the materials in the features, the very soft adjacent rock and the project goals, the decision was made to use jet grouting. In order to remove as much of the in situ detritus as possible, it was decided to use the three-fluid system. As noted by Burke et al. (1989), this system would help to quantify the amount of clay removal and, vitally, would ease slurry expulsion by reducing the slurry viscosity. Due to the very variable and unpredictable nature of the limestone and its features, it was not realistic to aim for total removal of all clay: the concept of providing grouted buttresses to minimize lateral movements of the fresh rock pinnacles during a seismic event was instead pursued.

Initially a treated soil strength of 3.4 MPa was set for features between El. 76 and 61 m but this was relaxed to 1.4 MPa beneath El. 73 m upon further seismic analyses.

Since the precise locations and depths of features were largely unpredictable, a systematic approach to treatment geometry was evolved. A regular distribution of holes was shown on the plans as a common basis for bidders (Figure 2). However, provision was made to install the columns in a Primary-Secondary-Tertiary sequence, with the exact locations of the higher order holes determined in the field based on progressive and continuing analyses of all the drilling and grouting data.

3.3 Execution

Work was conducted from the deck of a cofferdam driven around the footprint. The deck was at El. 104 m, the top of the sand at El. 95 m, and the average top of rock was El. 76 m. Water level varied with season.

The full-scale production works were preceded by a series of test sections (Column Confirmation Tests "CCT") to optimize procedures and verify their effectiveness (Section 3.4 below). In production mode, steel casings (324 mm o.d.) were rotated to rockhead or to a maximum elevation of 73 m. A 229-mm down-the-

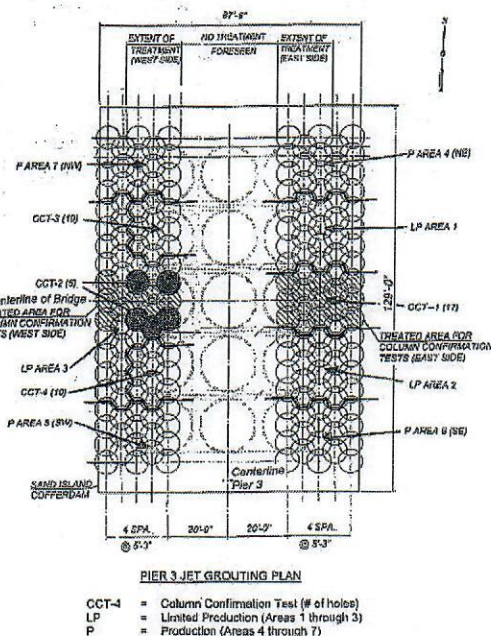


Figure 2. Conceptual grout hole layout, and CCT area locations, Cape Girardeau, MO.

hole hammer was then used to drill the rock to El. 61 m, the nature of the rock mass encountered being carefully logged. A 203 mm i.d. PVC pipe was then installed to rockhead and the steel drill casing withdrawn. This standpipe was intended to ensure easy travel of the drill spoils to the surface for observation and disposal. Each hole was then re-drilled with the jet grout rig equipped with a triple fluid monitor. The favored jetting parameters provided an applied specific energy of 250 MJ/m below El. 73 m and 330 MJ/m above El. 73 m. The grout had a water/cement ratio of 0.8 by weight, with superplasticizer at 1.25% by weight of cement (for fluidity), and hydrated bentonite at 3% by weight of water (to provide volume stability when injected into large features). During water and air jetting, a clay dispersant was used at 0.75% by weight of cutting water to enhance the efficient removal of clays and to ease the routine venting of spoils to the surface of the deck.

3.4 Control and Verification

Four phases of CCT testing were performed in structurally less critical areas of the footprint before sufficient confidence was generated in the processes and parameters to allow systematic treatment to commence across the whole foundation. A rigorous QA/QC plan was enforced to document and eliminate potentially significant construction variations. For example, columns that were grouted with poor spoils venting over large sections of feature were recommended for replacement. In total, 10 of the approximately 190 columns installed during the production phases were replaced for such reasons.

As part of CCT-1, pre- and post-grouting cross hole seismic studies were made. This provided strong evidence that low velocity zones (karstic features) had been transformed into higher velocity zones and that the treatment was providing general benefit to the foundation mass. Wet grab sampling was conducted routinely confirming compliance with the strength requirements. Verification coring of the treated mass was conducted as a final check, with a 90% core recovery criterion. Overall, 23 cored holes encountered an average of 56% limestone, 35% treated feature, and 9% clay. All except one of the cored samples met the strength criteria, although this sample was judged to be in a structurally non-critical location.

The authors believe that of all the construction factors influencing the quality and extent of treatment, those affecting the efficiency of the jetted spoils return during grouting were by far the most significant. In this regard, the proper selection of appropriate borehole and jetting tool diameters, injection parameters, and mix design (for both grout and jetting water) was critical.

The next construction contract has been let and construction of Pier 3 is currently underway on the treated foundation.

4. CONSTRUCTION OF HIGH CAPACITY MICROPILES (Major reference: Cadden, Bruce and Ciampitti, 2001)

4.1 Project and Problem Evaluation

It was planned to build a major extension to an existing shopping mall in Chester County, Eastern Pennsylvania. This expansion would triple its size to 111,500 m², and consisted of the addition of three department stores, a food court, and a second-story overbuild for the southern half of the mall. The maximum column loads were 667 kN for the stores and 4000 kN for the overbuild. Maximum column loads (for two new adjacent parking garages ranged from 1334 to 4450 kN.

The construction limitations of this project required both the engineers and contractors to provide innovative means for completing the work with minimal inconvenience to the shoppers and store owners. Restrictions included performing the work while most, if not all, of the stores remained open, and performing all the interior work at night when the mall was closed. Furthermore, the site was underlain by karstic dolomitic limestone into which a deep foundation system had to be installed to support the column loads.

Historical records indicated the area had experienced considerable recent sinkhole activity, and a quarry had also been developed in part of the site. Fill depths in this area reached over 8 m, compared with the 1.8 to 4.3 m of alluvium covering the remainder. Residual soils were locally found to extend to almost 25 m in the solution bedrock.

4.2 Design of Solution

Of the several systems considered, micropiles (Bruce and Juran, 1997) seemed best suited to the logistical

limitations, highly variable overburden thickness and rockhead elevations, and irregular rock mass quality. These drilled and grouted micropiles were required to have a unit service capacity of 1334 kN, and were designed as Case 1, Type A (gravity grouted) or D (posted grouted).

A total of 405 micropiles were required for the overbuild – 294 in the interior of the mall and the balance in the exterior – in groups of 3 or 4 per column footing. The foreseen lengths varied from 6.1 to 47.5 m (average 10.4 m) below existing base slab elevation. A further 355 micropiles were required for the parking garage in groups of 3 to 20 per footing. These micropiles varied from 7.6 to 54.9 m (average 13.1 m) in depth.

The piles comprised 178 mm o.d. drill casing ($f_y \geq 550$ MPa) with a 12.2 mm wall thickness, to transfer the load to competent rock. Therebelow, two structurally acceptable options were considered for the 3.1-m-long bond length (Figure 3):

- “traditional: – two 4.6-m-long, 57-mm-o.d. bars ($f_y = 517$ MPa) as central reinforcement, and extending up into the casing 1.5 m.
- “alternative” – extend the steel casing as the reinforcement into the bond length, but equip it with sleeve ports for pressure grouting via an internal double packer.

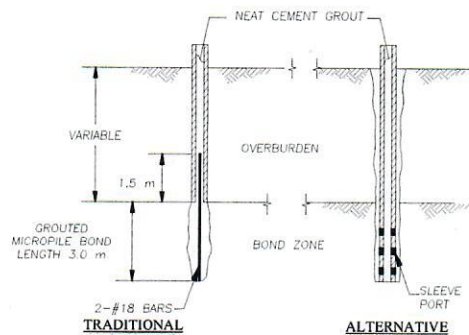


Figure 3. Schematic of micropile options, Chester County, PA.

It was reasoned that the alternative method would obviate the need for any separate phases of pre-grouting and re-drilling to provide adequate bond conditions in holes with difficult ground conditions.

Pile grout was to consist of Type I cement with a water/cement ratio of 0.45 by weight, and so a 28-day unconfined compressive strength of at least 28 MPa.

4.3 Construction

A pre-production test pile program was conducted to verify the practicality of the construction methods, as well as the ability of the installed elements to accept the required loads (Section 4.4 below). The “alternative” piles were first constructed, but routine drilling and grouting efforts were hampered by the presence of a localized “running soil” condition beneath the watertable.

More specifically, this phenomenon caused soil to pack around the sleeves and prevented them opening reliably during pressure grouting. The “traditional” method was then followed, allowing for pre-grouting and re-drilling of the bond zone where necessary. Drilling was conducted using rotary percussive (eccentric) duplex methods with air flush. Progress rates were far lower for the interior piles, where headroom restrictions necessitated the use of 1.5-m drill rod and casing lengths. Special environmental protection methods were required for these piles.

4.4 Control and Verification

A total of ten pre-production test piles were installed – five with each construction option. Of the “alternative” piles, Pile 1 could not be grouted (due to soil packing around the sleeves) while Pile 2 had a structural failure at the head during testing (eccentric loading problem). All the other piles successfully sustained at least the minimum test load of 2669 kN (twice working load), while two were loaded to the safe maximum stress levels of the reinforcement (up to 3638 kN). All piles were tested in compression in general accordance with ASTM-D1143, but modified with cyclic loading (PTI, 1996) to generate data relevant to debonding phenomena in the bond zone. Table 3 summarizes load test data.

Table 3. Summary of Micropile Compressive Load Test Data, Chester County.

Test Pile	Total Casing Length (m)	Bond Zone in Limestone (m)	Bond Zone Diameter (mm)	Max. Test Load (kN)	Max. Total Movement (mm)	Max. Residual Movement (mm)
3	10.6	3.0	218	3336	23	9
4	8.8	3.0	218	2669	13	3
5	10.4	4.6	218	2669	13	3
6	11.9	3.0	188	3638	23	6
7	30.5	3.2	188	2669	33	4
8	21.5	3.2	188	2669	48	Not recorded
9	10.7	3.0	188	2669	19	2
10	12.2	4.6	188	2669	33	2

The average rock/grout bond values at maximum test load ranged from 0.85 to 2.05 MPa (average 1.41 MPa). None of these were ultimate values, and justified the use of a value of around 0.7 MPa for the design working interfacial stress.

Thereafter, all the micropiles were successfully installed despite difficulties encountered during drilling and grouting that required care by the contractor to ensure consistent installation quality control of each pile. Full time observation and testing by experienced engineering inspectors were also imperative on this project to ensure that anomalous conditions were identified and that suitable steps were taken to establish confidence in each pile.

5. FINAL REMARKS

The design, execution, and verification of specialty geotechnical construction processes is always a particularly challenging task when engineering in and above karstic limestone terrains. However, the authors hope to have demonstrated that by employing the best practical concepts and processes at every stage of the project, high goals can be reasonably set, and successful results reliably achieved.

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