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Site Development in Deep Karst Terrain

Thomas P. Hart
Black & Veatch Corporation, Overland Park, Kansas

Giuliana Zelada-Tumialan
Black & Veatch Corporation, Overland Park, Kansas

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Thomas P. Hart, P.E., R.G.
Black & Veatch Corporation
Overland Park, Kansas

Giuliana Zelada-Tumialan, P.E.
Black & Veatch Corporation
Overland Park, Kansas

ABSTRACT

The development of a site for a power plant was complicated by the presence of pinnacled karst bedrock located beneath more than 24 meters (80 feet) of intermediate geo-materials (IGM). Shallow mat foundations had been planned in the conceptual design. Detailed subsurface investigations included an extensive supplemental drilling phase that identified a complex layer of pinnacled karst lying beneath a hard, nearly inelastic layer of residual IGM. In an effort to keep the site on shallow foundations, the design team attempted a preload approach to pretest the load-bearing capacity of the potentially karst prone site. Project schedule constraints required the preload be removed and abandoned in lieu of more positive, fast-track solutions after detection of collapse-like movements. With schedule milestones fast approaching, the project moved to conventional H-piles to expedite foundation construction. The pile design effort included an indicator pile program with confirmation borings, a static load test and PDA testing. Pile installations were installed up to a depth of 101 meters (332 feet) and rejections penetrated over 125 meters (410 feet). Predicted averages for the project were met at 30 meters (100 feet). Project evolution included settlement plots for 28 shallow settlement plates and three deep anchors reflecting not consolidation, but small shear movements as the overburden crushed the underlying pinnacles.

BACKGROUND

New power demands worldwide often require development of new generating stations. Site selection for new generating stations is dictated by numerous factors and considerations, not the least of which may, perhaps be geology and topography. Power producers often look for unpopulated or industrial areas, near intersections of existing transmission and fuel supplies (gas, oil or rail/coal), and near a generous, available water supply. Site issues such as floodplains, seismicity, expansive soils, topography, and karst conditions are often economically balanced against transmission, fuel and water issues within the larger scheme of developing a new generating station.

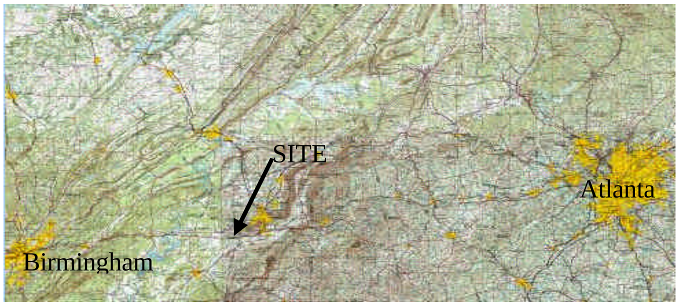

Figure 1
Site Location

In late 2001, Black & Veatch was selected to provide engineering, procurement and construction for a new power plant in northeast Alabama. The new generating station would provide northeast Alabama and many new developments, such as the new Honda Plant in nearby Lincoln, a dependable supply of electricity. The subject site is located in southwest Calhoun County, Alabama, between Birmingham and Atlanta.

The southern end of the Appalachian Mountains around Birmingham is well known for karst activity. The problems with karst development in the Birmingham area are well documented by George Sowers. Most notably and invaluable to our work was the publication "Building on Sinkholes," by George Sowers, published by ASCE (1996). What is not well referenced is the areal extent of the karst prone area. The unfortunate reality in underdeveloped, rural areas is that construction encounters with karst are rarely if at all documented, and can only be discovered through conversation with local residents and contractors. A local Birmingham area engineer who was interviewed during the investigation had no knowledge of the karst terrain extending this far east of Birmingham. As the project moved forward, more information about encounters with karst in the area began to surface, including the nearby Honda Plant, Telledega Superspeedway, and an Anniston shopping mall parking structure.

INVESTIGATION

The preliminary investigation was planned to provide information to support foundation choices and support the development of the cost of foundations and grading. There was no prior knowledge of soil or bedrock conditions other than the County Soil Survey and the State Geologic Map. Preliminary review of existing literature showed the site was located within the Valley and Ridge physiographic provence of the Appalachian Highlands. Local bedrock was identified as Ordovician and Cambrian aged dolomite and limestone.

The field investigation included 10 conventional SPT borings planned to depths of 21.3 meters (70 feet). Preliminary investigations typically estimate boring depths by the 2B rule of thumb, plus some additional length for good measure. Given the upland topography, we anticipated encountering bedrock at some point prior to the planned depth. The largest foundation for this type of plant is the combustion turbine (CT) foundation, typically about 7.6 meters (25 feet) wide. The field investigation also included piezometers, soil resistivity tests and test pits.

There were a few surprises coming out of the investigation. First, the depth to bedrock was greater than anticipated and second, the overburden soil was stiff to hard residual intermediate geo-material (IGM) throughout the borings. The biggest surprise was finding a soft pocket of soil, just above bedrock in two of the ten borings, outside of the power block where the major foundations are located. The soft zone was at a depth of over 25 meters (85 feet)! Here lies the great chasm between theory and practice. Will a soft zone beneath 25 meters of IGM effect shallow foundation design? Our conclusion was that our 7.6 meter wide foundations would not be affected when following conventional consolidation theory. But what about karst terrain and future sinkhole development or collapse?

The final field investigation attempted to comprehensively probe the limits and extent of karst at the site. Thirty-three additional borings were advanced to refusal at the site with bedrock cores collected in eight of the borings. What was found completely changed our opinion about the suitability of shallow foundations at the site. The site was underlain by severely pinnacled limestone.

The investigation started off in dramatic fashion with the first boring encountering much softer clay from the ground surface through the entire IGM zone, beneath one of the heaviest, most critical pieces of equipment. The rest of the borings were a mixed bag of strong IGM overlying either bedrock or soft clay. The top of bedrock was defined at depths ranging from 10 meters (33 feet) to 40 meters (132 feet). Borings extending beyond 21 meters found the signature soft, normally consolidated clay typically found in slots between pinnacles of limestone. The thickness of the slot material varied from very thin to thicknesses of 15 meters (50 feet) or more.

RISK-BASED SOLUTIONS

We now had to address the uncertainty of the karst behavior over the life of the power plant. We could not find a reference of a deep karst site in residual upland environments that had seen ravelling or collapse. Sites with pinnacle or karst development at this depth are rare in the Appalachians.

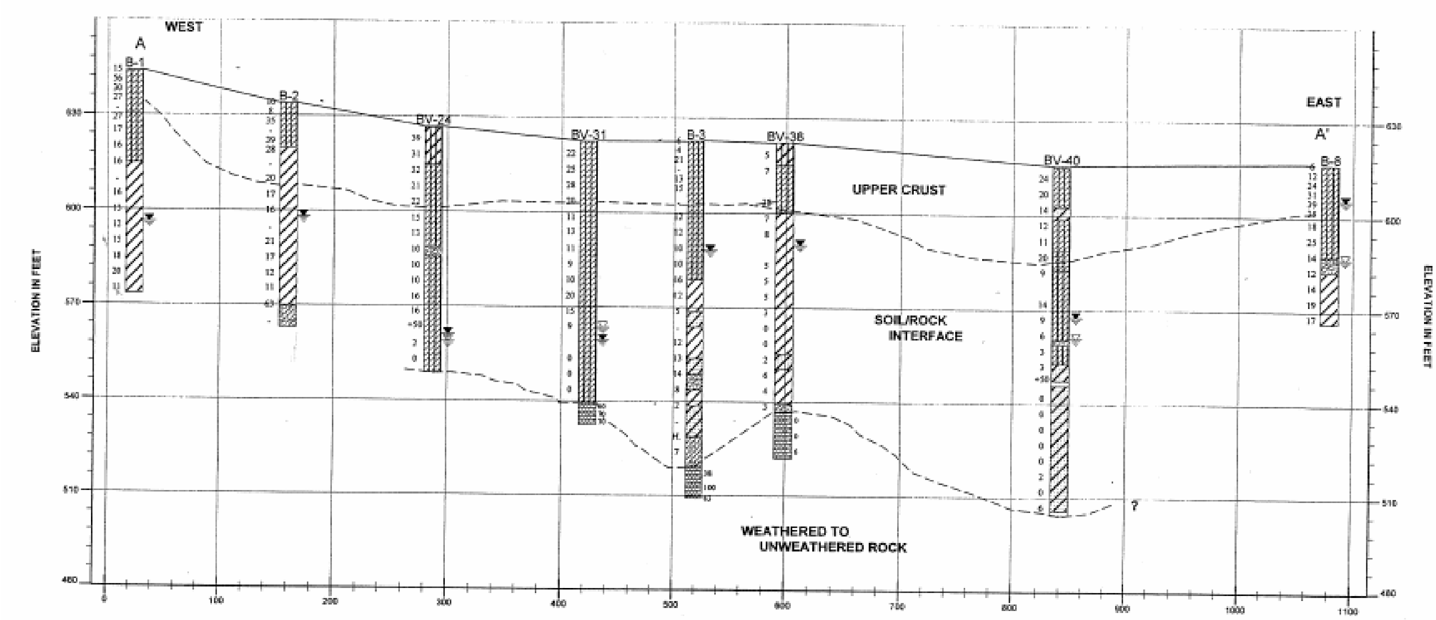

Figure 2 East-West Cross Section

Sowers suggests using pre-loads to simulate the weight of the foundations. If the pre-load was too heavy for the pinnacled karst system to support, the overburden would crush the pinnacles or settle over the pinnacles until stable. The entire process would occur faster and more immediately than typical consolidation theory would suggest, but just how fast was unknown. The project went ahead with plans to preload the site with 4.6 meters (15 feet) of pre-load, on top of an already cut and filled plant site. Maximum thickness of fill, including the preload, would reach nearly eight meters toward the east end of the site. If the pre-load triggered sinkholes or caused significant settlements, the project would review other options, including cap grouting or deep foundations. Local wisdom shared by engineers in the Birmingham area warned that both grouting and deep foundations were fraught with risk. Grouting could interrupt drainage, triggering collapse. Deep foundations costs could be difficult to control with uncertain and extremely variable depths to bedrock.

CONSTRUCTION

Pre-Load

The pre-load was scheduled for three months. The monitoring plan incorporated both conventional settlement plates installed immediately beneath the preload, and borros anchors, drilled in and installed 25 meters below the plant grade. The shallow plates would reflect an overall picture of settlements across the site. The deep plates would identify how much settlement was occurring in soft clay between the limestone pinnacles.

Predicting the amount of settlement from consolidation tests was determined by what was thought to be fairly conservative measures. Residual clays in the upper 20 to 25 meters crushed Shelby tubes, and the gravel content of the clay made quality sample selections very difficult. As a result, only the softest soils at depth were characterized for consolidation characteristics. The consolidation parameters from deeper soil were used to characterize the entire overburden thickness, resulting in a worst-case consolidation settlement prediction.

The early settlement readings were variable. Some areas showed no movement for weeks. Others would settle one week, stop moving for several more weeks, only to resume movement later on.

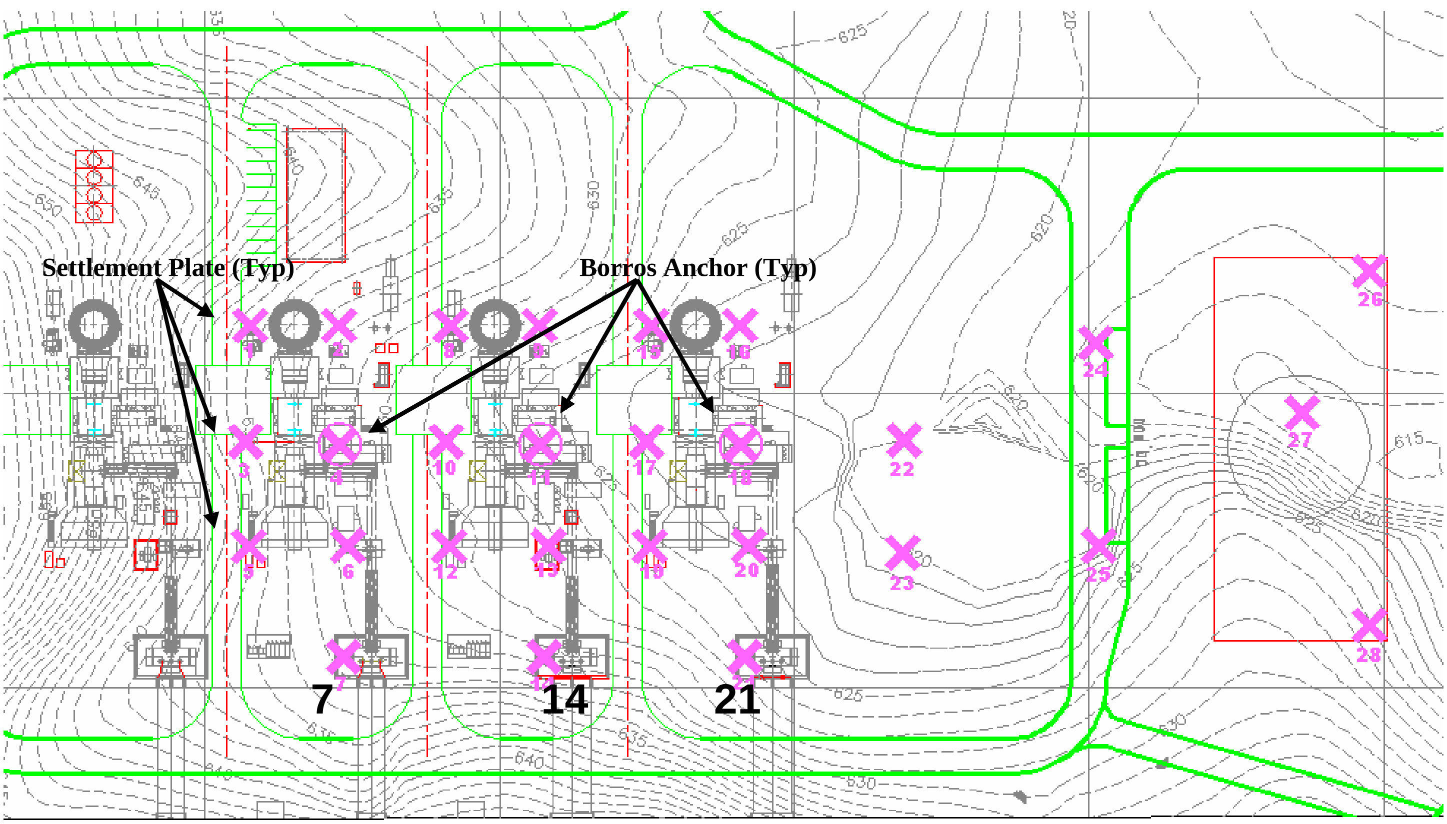

Figure 3: Settlement Plate Plan

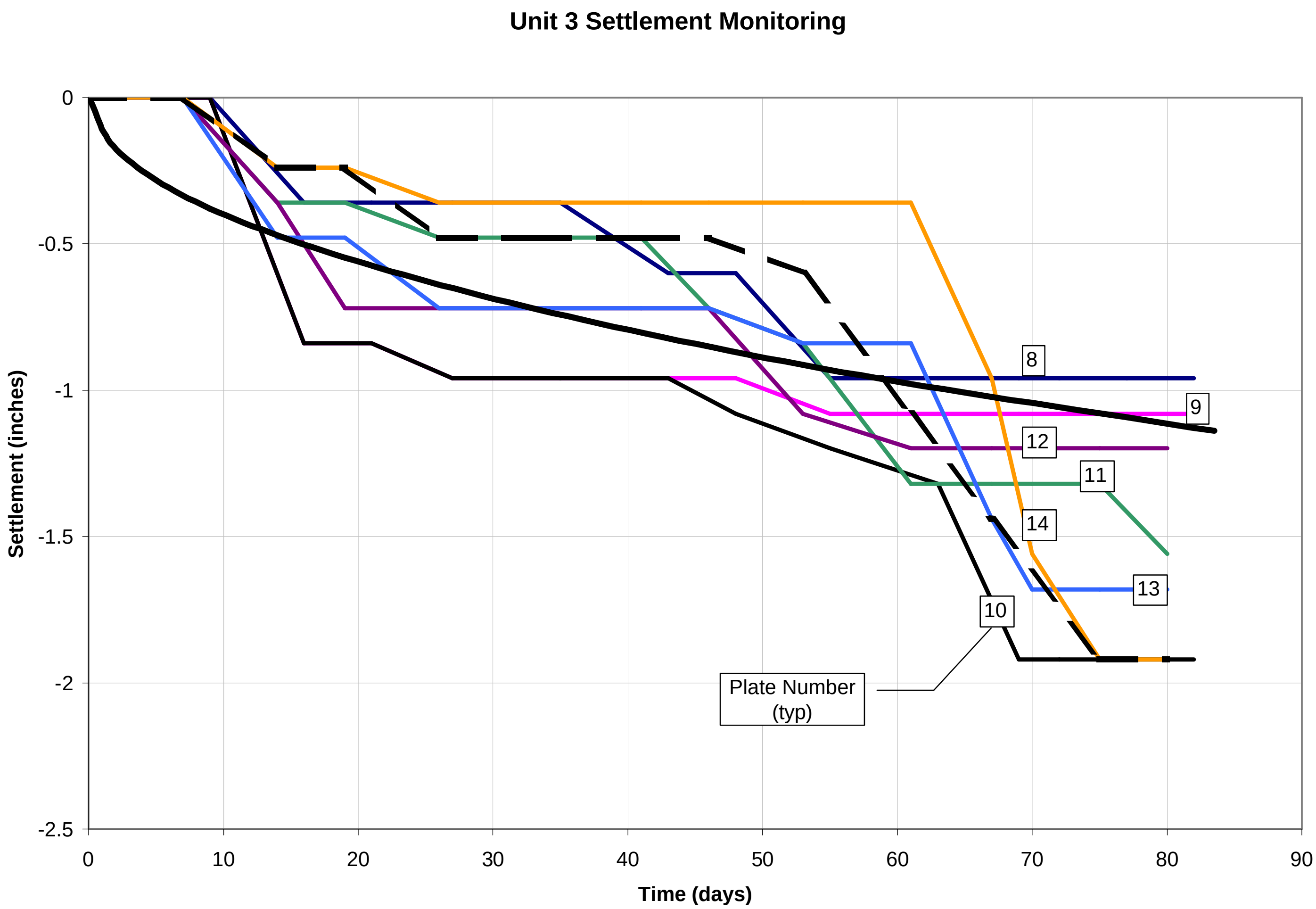

Figure 4. Illustrates data from 8 settlement plates beneath a CT/transformer foundation. Settlement data reflects characteristic recurring movement and stabilization reflecting the crushing behavior of the overburden. The dashed deep plate, anchored just above bedrock reflected compression of the soft soil between pinnacles. Stability was reflected in 23 of 28 plates at 70 days. At 75 days, 3 of the 7 plates at Unit 2 (Not shown, westernmost foundation on fill) resumed movement, as well as plate 11 seen here.

The settlement data (Figure 4) for the foundation near the center of the pre-load illustrates the chaotic behavior of the karst regime. The predicted consolidation is shown as the smooth, hyperbolic solid line. The deep borros anchor is the dashed line. Both shallow plate and deep anchor data illustrates the stop-start settlement behavior.

At 75 days, the data appeared to be stabilizing. Plans were underway to begin stripping preload at the west end of the pre-load. At 80 days, preload removal had begun when the final set of readings were collected. However, unexpected news soon followed: several plates beneath the west and center foundations had begun moving again. The stripping of pre-load was stopped. Predicting the remaining duration of the settlement and stabilization proved impossible. Difficult decisions had to be made quickly to maintain the construction schedule.

Early discussions with compaction grouting contractors turned up no guarantees. Grouting contractors were expensive and unwilling to warranty the results. The project turned to steel H-piles to reduce risk to the major foundations and to maintain schedule. The uncertainty of quantities were managed during construction.

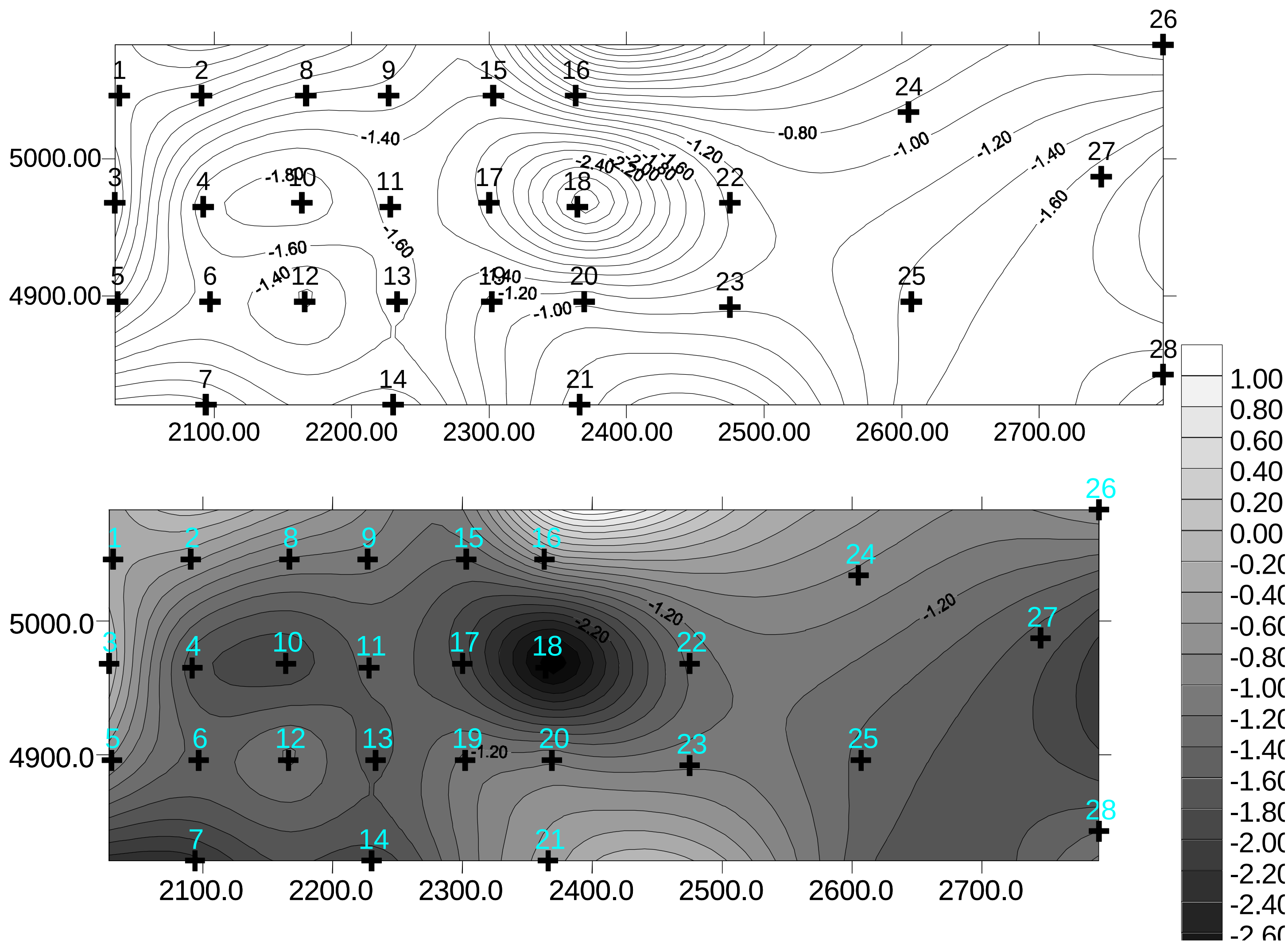

Figure 5. Final (total) settlement contours (distance in feet, settlement in inches). Areas of the greatest settlement magnitude at the surface grade are shaded dark (bottom). The circular nature of the concentrated settlement may reflect karst movement with maximum settlements over six cm. Pile driving records reflect very deep cavities in the bedrock near settlement point 18 with piles reaching depths of over 60 to 90 meters (200 to 300 feet). Conversely, the relatively stable area near settlement 21 reflected very little movement, but piles reached the greatest depths on the project, with one rejected at a length of 124 meters (407 feet) and some meeting solid refusal at 93 to 101 meters (305 to 332 feet).

RAPID DESIGN

The construction schedule drove the foundation design process. With major foundation construction scheduled to begin in a few weeks, Black & Veatch engineers set out to interview and hire a pile driving contractor, the Morris Group of Birmingham. Pile selection was determined by the availability of materials. A large quantity of HP 12X63 pile happened to be available. Pile tips would be protected using APF Hard-Bite pile points. Pile design capacity would be determined by results of a static load test and dynamic load tests during an indicator pile program.

<u>Indicator Piles</u>

The design objective was to achieve a 534 KN (60-ton) pile with capacity developed through end bearing on sound limestone, located at an anticipated average of 100 to 110 feet beneath the site. The capacity would also have to account for downdrag.

Within two weeks of the decision to use piles, a four-pile

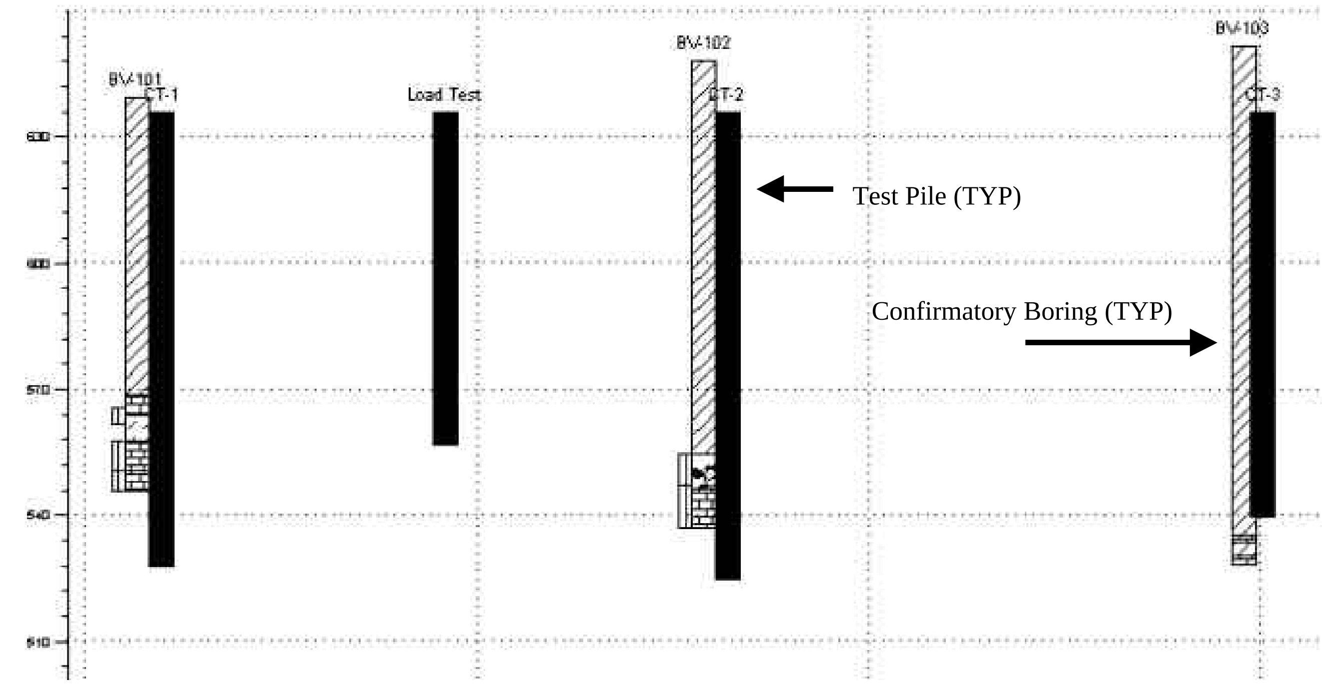

Figure 6. Indicator piles shown as solid black lines in elevation, over confirmatory offset boring, represening the variability in the top of rock over very short horizontal distances. HP 12X63 piles refused both above and below where confirmatory borings would indicate.

indicator test program was underway. The indicator piles were driven under each of the four major foundations. First, four more borings were drilled at the indicator piles locations to determine the depth to competent rock. Refusal criteria and ultimate capacity were developed while driving the piles using a pile driving analyzer (PDA). A full-scale static load test confirmed the results. A refusal at five blows per 1.25 cm (five blows per ½ inch) resulted in the desired allowable capacity of 534 KN (60 tons).

Pile driving hammers selection also proved difficult. The most widely used hammer in the Birmingham area, the Delmag D19-42, was mobilized to the site for the indicator program. It was quickly apparent the 54 KN-m (48,000 ft-lb) hammer could hardly penetrate the dense IGM overburden requiring between 50 and 100 blows per 30 cm (50 to 100 bpf) from the ground surface to refusal. The Morris Group switched to a Delmag D30-32 (102 KN-m/75,000 ft-lb), significantly increasing the rate of penetration to rough average of 15 to 30 blows per 30 cm.

The results of confirmatory drilling were mixed. Piles were driven to refusal far deeper and shallower than borings offset only a few feet, confirming the extremely jagged nature of the pinnacled limestone surface (Figure 5). Using the average lengths of the indicator pile, Black & Veatch estimated the quantity of pile for the project at nearly 12,200 lineal meters (40,000 linear feet) with an average length of 100 feet.

The overall duration between the decision to switch the design to deep foundations and the completion of the test program was two weeks, with full scale production beginning in just three weeks. The demands of the construction schedule were met.

<u>Production Piling</u>

Pile behavior was typified by hard driving followed by abrupt refusal. A smaller portion of piles exhibited a decreasing resistance for some distance above refusal.

Definition and determination of rejected piles remained a work- in-progress throughout the pile driving program. The behavior of the piles while driving remained relatively consistent through the upper 20 to 25 meter (70 to 80 foot) crust. The upper crust did not reflect the typical erratic behavior exhibited by suspect piles below 20 to 25 meters. Piles that were advanced significantly beyond adjacent pile completions were flagged as suspect. Re-evaluation of each suspect pile was repeated upon subsequent splices. All of the rejections were based on length or excessive lean (batter). Consideration was given for these long piles and their location in the mat, as well as the type of foundation. Piles in the CT mat were required to take dynamic machine loads, and were held to a tighter rejection length than those in static foundations (exhaust stacks, transformers and fuel tank). Piles were also evaluated based on their adjusted spring constant and whether the mat could handle the spring constant variability.

When the major foundations were completed, the estimated average length of pile from the indicator program proved accurate. The completed average pile length for the 357 production piles was only 2 inches longer than predicted. The final rejection rate was just over 8%, far less than the predicted 15%. Piling predictions and quantities for the fuel tank (located at the far east end of the site, where the worst predicted conditions were assumed) were far less favorable. The completed average pile length for the fuel tank jumped up to 109 feet. The rejection rate increased to 11%, still significantly less than the 15% prediction.

CT - 4

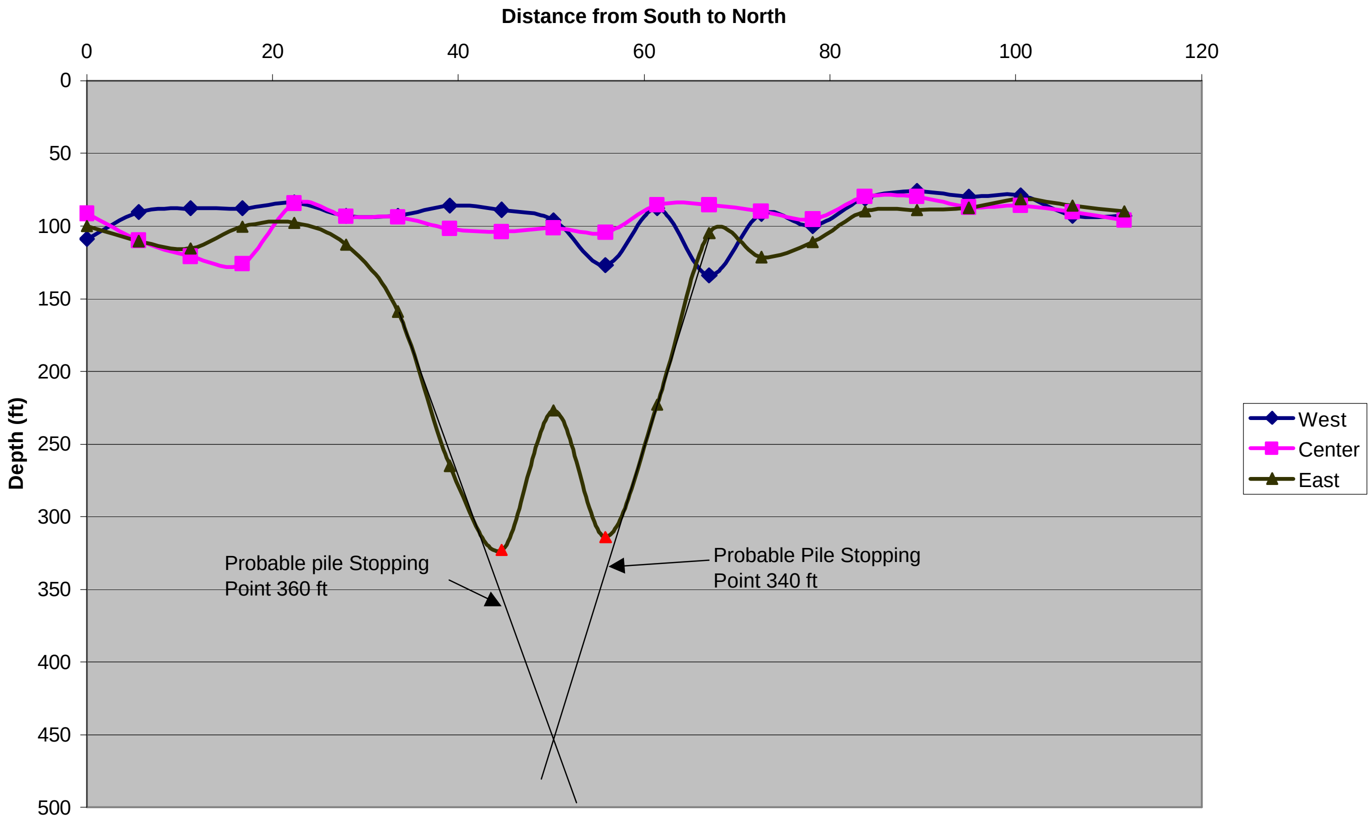

Figure 7. Pile tip refusals for three rows of piles. Each row is two to three meters apart. Illustrates the large settlement feature reflected in Figure 5. The deepest two piles were abandoned for excessive length.

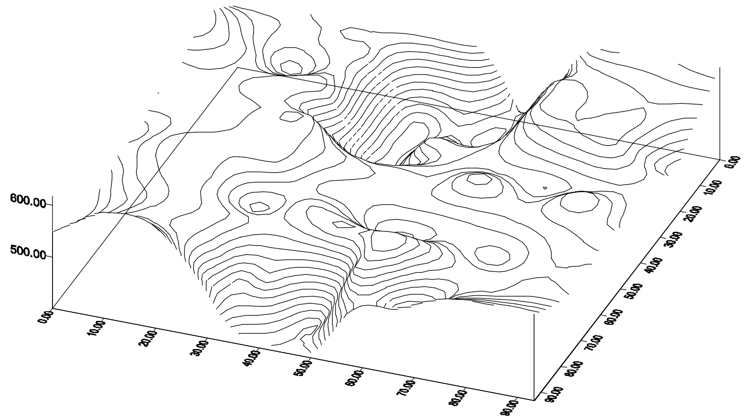

Figure 8. Surface of limestone bedrock, generated from fuel tan area at the east end of the site.

Discoveries and Conversation

Tools, such as conversation with property neighbors, would be very useful to the design process, but is largely impractical since design investigations are often pre-land-purchase and information regarding developments is usually confidential.

During the pre-load period, the owner's project manager walked along a potential fuel pipeline corridor, north of the site with the adjacent property owner. When coming across dry pond, the property owner expounded on how he could not seem to plug the large holes in the bottom of his "pond." Upon further reconnaissance and inspection, we discovered no tell-tail signs of an excavation. No dam or spoils pile were evident, very unusual characteristics for an old pond. The dry pond appears to be a very large, 30 meter by 60 meter sinkhole, three to five meters deep, with several one half meter to three meter diameter throats daylighting along the base. The probable sinkhole is more than double the size of the largest foundation planned on site, and was located in dense woods, approximately 30 meters north of the property line.

The neighbor bordering the south side of the site reported his ponds "flushing" right before his eyes some years prior to this project. The neighbor clearly recalled watching his prized fish disappearing into the cavern. The resourceful man borrowed a sheeps-foot roller and proceeded to plug the hole with compacted clay, thereby successfully rehabilitating his pond.

Figure 8. "Dry Pond" located on adjacent property, about 30 meters north of the site.

As time went on, more shallow depressions became apparent in the surrounding countryside, visible only as temporary ponds, which would appear in fields after large rainfall events and quickly disappear over the next few days. These shallow depressions do not have enough relief to appear on typical U.S.G.S topographic maps.

Figure 9. *Karst throat inside the dry pond.*

CONCLUSIONS

Great caution should be exercised when investigating sites with underlying carbonate bedrock. If preliminary borings had stopped in the hard overburden crust at a depth equal to two times the foundation width, the karst conditions would have gone undetected. Some portion of any karst site investigation must examine the depth and condition of the bedrock. Great care should be exercised when proceeding with foundation design.

Driven pile foundations would not be the first choice given such deep karst conditions. However, pile foundations can be successfully driven in Deep Karst Terrain through hard IGM soils. Quantities can be reasonably predicted and controlled using tried and true indicator pile test programs coupled with static and dynamic load testing and a well defined refusal and rejection criteria.