Case History Illustrating the Challenges of Foundation Design and Construction in Karst Terrain

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ABSTRACT

This paper discusses the challenges associated with design and construction of foundation systems for a Corporate Campus located in Chester County, Pennsylvania that is underlain by Karst terrain. A comprehensive subsurface investigation was implemented to develop adequate foundation systems and related site work precautions. Because there was evidence of sinkhole activity prior to any construction work, and the subsoils revealed some variability from a consistency/density standpoint, the selected foundation system design included a combination of soil improvement using compaction grouting for shallow foundations and deep drilled-pier foundations. After construction activities began, several occurrences of solution activity were documented and repaired. During construction of drilled-pier foundations at one of the structure locations, a significant number of voids and discontinuities in the rock were encountered. The impact of these discontinuities and voids was dramatic to the effort and time necessary to complete the drilled pier foundation construction for this structure. After careful consideration of potential cost and schedule impacts, the foundation design for remaining structures was modified to eliminate the use of drilled piers and incorporated only compaction grouting for support of shallow foundation systems. Interaction of the Geotechnical Engineer, Construction Manager, Owner, and Contractor began early in the construction process and this interaction became critical to the project success as work proceeded on this project.

INTRODUCTION

SITE-Blauvelt Engineers, Inc. (SBE) was retained as the project Geotechnical Engineer for a Corporate Campus located on an 80-acre site in Chester County, Pennsylvania. Three office buildings (referred to as Buildings A, B and C herein), a parking garage and associated facilities were proposed for 17 acres of the 80-acre property.

The regional limestone geology underlying the Chester Valley is well documented and notorious for sudden and sometimes catastrophic sinkhole action. Although some limestone formations are more susceptible to sinkhole activity than others, it was believed that the potential for sinkholes at this site was as likely as any other in the Chester Valley, particularly due to the history of sinkhole formation both on the site and close to the site. This required that a comprehensive evaluation be implemented to develop adequate foundation systems and related site work precautions. Because there was evidence of sinkhole activity prior to any construction work, and the subsoils revealed some variability from a consistency/density standpoint, the foundation system design selected and implemented included a combination of soil improvements for shallow foundations and deep drilled-pier foundations. While these soil improvement and foundation systems are not unique, they do represent significant additional effort and cost compared to that necessary for non-sinkhole prone areas, and exemplify the level of precaution that is necessary to adequately and safely support structures in these areas.

Site Characteristics and Features

The site is generally hilly with occasional steep and severe slopes and rock outcrops. Surface drainage is generally in the northern direction. Small tributaries exist on site in the southwest and central portions of the property. These tributaries may be considered headwater streams to a creek that runs through the project site. An abandoned railroad embankment is located north of the building areas. The tributaries flow through a culvert below a roadway embankment.

Proposed Construction

The construction consisted of three, three to four-story office buildings, one multi-level parking garage, and related infrastructure. Of the roughly 80 acres available on the property, less than 17 acres were developed for this project and approximately 63 remain undeveloped. Table 1 summarizes attributes of the structures.

Construction of the office buildings consisted of conventional steel beam, girder, and column framing, with column bays that

Table 1. Summary of Structure Attributes

Structure	Stories	Footprint Area (ft^2)
Building A	3	34,000
Building B	4	39,700
Building C	3	38,800
Parking Garage	3/4	60,700

ranged between 30 by 30 feet to 30 by 40 feet. Typical interior column loads are 300 to 390 kips dead load and 120 to 150 kips live load. Typical exterior column loads are 240 to 300 kips dead load and 72 to 90 kips live load.

The parking structure is an "open" parking structure, consisting of structural precast concrete T's for the decking, and cast in-place bearing walls. Typical interior column loads are 360 to 450 kips dead load and 150 to 190 kips live load. Typical exterior column loads are 240 to 300 kips dead load and 72 to 86 kips live load. The interior bearing walls featured dead loads of 9 kips per foot and live loads of 5 kips per foot. Allowable differential settlements for the office building and parking garage were limited to ½ in. between adjacent columns.

Regional Geology

Published geologic data indicates that the site is underlain by the soils/rock of the Conestoga Formation. This formation consists of micaceous limestone, phyllite, and alternating beds of limestone and dolomite. The Conestoga Formation generally strikes north 65º east and dips steeply to the south. The limestone and dolomite rock commonly form irregular pinnacles and occur in random fashion throughout any particular area. Due to solution activity within this formation, subterranian boulders are common.

The natural surficial soils directly below the topsoil consist predominantly of low-permeability silts and clays with varying amounts of sand formed from the in-place weathering of the underlying parent rock. The formation of the mantle soils in a limestone geologic setting similar to this site inherently leads to very fine-grained silty/clayey soils near the surface becoming less fine-grained and more structured with depth. Because the soils closer to the surface are more weathered and fine grained, they have much lower permeability characteristics than the underlying less-weathered coarse-grained soils. Where phyllite derived soils are present, the soils below the silty/clayey surface mantle soils are commonly micaceous silty sands. In areas where no phyllite soils exist and limestone-derived soils are present, the subsoils are predominantly silty/clayey materials that extend from the surface to the top of rock.

The geology underlying the hills south of the site include residual soils and bedrock of the Wissahickon Formation, a heterogeneous regional metamorphic bedrock unit. The residual soils are typically highly micaceous sands and silts increasing in density with depth and gradually transitioning into completely to highly weathered rock. The rock of the Wissahickon Formation is a heterogeneous regional unit of schists, gneisses, and sometimes phyllites. The predominant minerals found in the Wissahickon schist are quartz, muscovite, feldspar, biotite, and chlorite. The most recognizable and abundant component is white muscovite or mica. Folding and faulting episodes throughout regional geologic history have produced extensive foliation in the Wissahickon schist. Weathering profiles in the mica schist also tend to be highly irregular.

Documented Sinkhole Activity

As mentioned previously, geologic formations in the site vicinity are prone to sinkhole activity. Evidence of such activity existed at the site prior to the commencement of any construction operations. The existing sinkholes were considered long-term, possibly ancient occurrences of solution activity. One sinkhole was located on the northern slope of the hill south of Building B. This sinkhole was approximately 30 feet in diameter and 15 feet deep and was actually mapped as a topographic depression on the original survey mapping for the site. Another sinkhole was located at the northern edge of the proposed detention basin at Building A. This depression was irregular, about 20 feet long and six to eight feet deep. It is noted that the location of these sinkholes was not in areas of previous concentrated surface water flows. Sinkholes were also prevalent in the eastern portion of an adjacent corporate site located to the west of the site.

Figure 1 presents the location of sinkholes as reported by the Pennsylvania Geologic Survey as well as critical sinkholes observed onsite. Of note is the trend of sinkhole activity observed in this area. The drawings clearly show a southwest to northeast trend of sinkhole activity through the project site that generally follows the strike of the formation.

SUBSURFACE EXPLORATION PHASE

Several investigations were conducted on this project site during the period from June 1985 through August 2000 as discussed below.

Previous Field Investigations

The 1985 investigation was conducted for a different client that was considering developing the site as a warehouse/office park of one or two-story buildings that occupied a substantially larger portion of the site than the current development. Five test borings and eight test pits were completed at the project site. Although this project was abandoned, the preliminary geotechnical investigation work was applicable to later development investigations.

A preliminary geotechnical investigation was conducted for the owner of the current development in 1994. This investigation was based on four office buildings, one parking garage, and three on-grade parking lots. One building and the associated parking

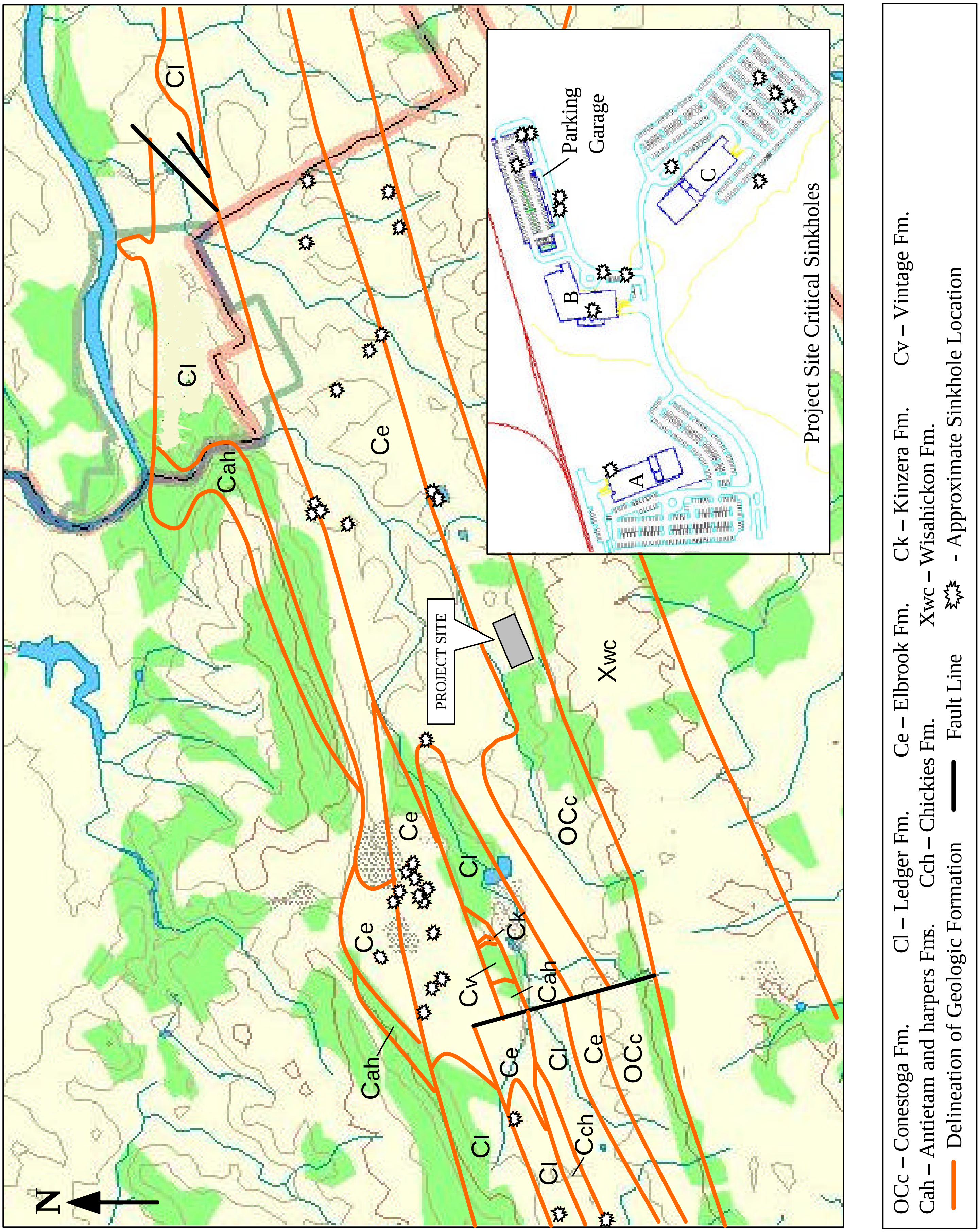

Fig. 1. *Sinkhole Location Map*

lot were later deleted from the program. This more extensive investigation included 27 borings and 14 test pits. The work was focused on development of preliminary design parameters for planning and estimating purposes.

Focused Site Investigation (for Current Development)

In 1999, supplemental geotechnical work was conducted to further define the geotechnical design parameters for this project. Thirty-four borings were drilled by our drilling division in August. Test borings were completed using conventional drilling equipment for the advancement of soil borings and rock coring. The boring locations were selected based on the locations of proposed building corners.

Test probes, performed using compressed air percussion drilling techniques, were proposed and executed as an efficient and cost-effective means of supplementing the information obtained through test boring. The original proposal included an estimated 116 test probe locations. This included a test probe at each column location where drilled piers or footings on rock were anticipated based on the results of preliminary subsurface investigations and at 60 percent of column locations where in-situ site improvement methods in conjunction with spread footings were anticipated based on the results of preliminary subsurface investigations. Based on budgetary constraints, the Client authorized execution of 47 test probes, which were also performed in August 1999. Locations were selected based on available test boring information and anticipated column locations.

Due to complications arising during construction of the site structures, supplemental geotechnical investigation work continued after construction activities began in order to refine design parameters and provide real-time data for construction purposes. In total, approximately 150 test borings, 200 test probes and 20 test pits were conducted to define various engineering characteristics of the subsoil and rock at this site.

DESIGN PHASE

Test Boring Data Evaluation

The results of the subsurface investigation revealed that the site is underlain by generally fine-grained materials, i.e., silt and clays, with varying amounts of rock fragments. Much of the soil materials encountered is a result of in-place weathering of the phyllite rock. The soils were encountered at depths varying from four to greater than 80 feet below existing grades. These soils were found to be generally "firm" to "stiff" with "soft"/"very loose" sandier zones immediately above the rock surface. This stratigraphy is common in limestone areas.

Underlying the soils is either limestone or phyllite rock. The phyllite was completely to severely weathered and soft. The limestone was moderately to slightly weathered and moderately hard to hard. The following paragraphs summarize the subsurface conditions encountered at each building location.

Building A (Finished Floor Elevation (Elev.) 248). The maximum cut and fill for this structure are seven and two feet, respectively. This area is predominantly underlain by soft, decomposed phyllite. Typically phyllite recoveries were less than 20%, which is indicative of very severely weathered soft rock. Some intact limestone was encountered in three test borings in the southern portion of the structure; however, voids were encountered in one of these three borings between elevations 229 and 217. Top of Rock varied from a high of Elev. 230 in the southern portion of the structure to a low of less than Elev. 163 in the northern portion of the structure. Many of the borings did not encounter competent intact rock within 80 feet of the ground surface.

Building B (Finished Floor Elev. 242). The maximum cut and fill for this structure are nine and 13 feet, respectively. This area is predominantly underlain by relatively intact limestone. Highly weathered soft phyllite was encountered in three test borings in the southern portion of the structure. Voids were encountered in two test borings in the northeastern portion of the structure between elevations 221 and 204. Top of Rock varied from a high of Elev. 240 in the northeastern portion of the structure to a low of Elev. 144 in the southern portion of the structure. The average rock elevation was 217.

Building C (Finished Floor Elev. 276). The maximum cut and fill for this structure are 10 feet and 14 feet, respectively. The structure is underlain by the most consistent intact limestone rock on the project site. The area is predominantly underlain by relatively intact limestone with rock recoveries varying from 40 to 100%. A void was encountered in one boring between elevations 254 and 252. Top of Rock varied from a high of Elev. 275 in the southeastern portion of the structure to a low of Elev. 213 in the northwestern portion of the structure. The average rock elevation was 253. Ground water readings indicate that the loss of drill water was not common. This is indicative of relatively coherent rock with few and/or closed fractures.

Parking Garage (Finished Floor Elev. 242). The maximum cut and fill for this structure are 18+ feet and three feet, respectively. This area is predominantly underlain by limestone. Highly weathered soft phyllite was encountered in four test borings which cover approximately 30 to 40% of the building area in the northeastern and southwestern portions of the structure. Voids (between elevations 201 and 198) and very soft soils (between elevations 218 and 214) were encountered in one boring in the northeastern portion of the structure. Top of Rock varied from a high of Elev. 254 in the southern portion of the structure to less than Elev. 148 predominantly in the northern portion of the structure. The average rock elevation was 221.

Foundation System Evaluation and Recommendations

Highly variable subsurface conditions were encountered at the project site. Various foundations schemes were investigated in view of the potential sinkhole development and the settlement criterion. In general, where sound intact rock is near the ground surface footings could be situated directly on the rock surface. A soil exchange could be implemented where rock is not excessively deep. Where intact limestone rock is at limited depth of 20 to 60 feet, drilled piers could be considered. At those locations where intact competent rock was very deep, spread footings in conjunction with compaction grouting was feasible.

Several options were investigated for foundations of the proposed structures. They are as follows:

1. Footings Situated in Rock. Where rock is shallow, i.e., within six to eight feet of finish floor elevation, foundations consisting of footings situated directly on rock were considered. This foundation option had limited applicability since rock is relatively deep at most locations but was utilized where possible since it is the most cost effective and stable foundation option.

2. Soil Exchange. Where rock is within approximately eight to 14 feet below finish floor elevation, a soil exchange down to intact competent rock was considered. This scheme would involve the removal of the overburden soils to competent rock within and 10 to 15 feet beyond the building footprint and replacement with structural compacted fill (i.e., PADOT 2A aggregate or flowable flyash fill). This scheme would marginally improve subsurface conditions with respect to possible sinkhole formation.

3. Compaction Grouting. Where rock was deeper, i.e., greater than 10 to 14 feet below finish floor elevation, compaction grouting of the subsoils to the rock surface or to a maximum depth below bottom of footings was considered. The depth of compaction grouting would be determined such that arching effects and the vertical stress distribution below the footing are considered. Grouting could be waived at some locations where continuous "dense" or "stiff" soils are encountered based on the results of supplementary test boring activities.

4. Drilled Pier Foundations. Where sound intact rock is predominantly within 20 to 60 feet of the finish floor, consideration was given to supporting the structure on drilled piers with rock sockets. Drilled pier design would include a combination of skin friction and end bearing, as use of predominantly end bearing is not recommended due to variability of rock quality and quantity. Properly implemented, a drilled pier foundation provides the best protection against loss of support due to sinkhole activity.

Details regarding foundation system selection for each building area are presented below in the order that they were constructed.

Foundations for Building A. Based on the proximity of the rock surface which had the potential to be in excess of 90 feet and the extensive phyllite encountered at the majority of the borings, footings on rock, a soil exchange, and drilled piers were not considered to be feasible at this structure. We recommended that this structure be supported on footings in conjunction with compaction grouting at all column locations.

Foundations for Building B. Based on the proximity of the rock surface, footings situated directly on the rock surface were feasible at a small northeast portion of the structure (approximately 10% of building area). Either a soil exchange or drilled piers was feasible in the north-northeast wing of the structure (approximately 50% of building area). In the southern wing of the structure, where rock is deeper (up to 98 feet below finish grade elevation), drilled pier foundations would be required.

The selected foundation design for all column locations at Building B consisted of a 36 or 60-inch diameter drilled pier supported primarily by rock sockets in intact limestone or phyllite. Rock sockets were recommended for 68 of the 72 column locations; recommended socket lengths varied in length from one to 12 feet, based on the anticipated depth to rock as well as the anticipated type and quality of rock at that location. The pier load resistance design was developed through a combination of skin friction and end bearing in the weathered limestone. The design neglected skin friction in the overburden soils due to the potential for loss of soil support from sinkhole activity. Table 2 below summarizes drilled pier design recommendations for Building B.

Table 2. Summary of Drilled Pier Design Recommendations for Building B

	36-Inch Diameter[1]		60-Inch Diameter[2]	
	Total Length[3]	Socket Length	Total Length[3]	Socket Length
Minimum (ft)	3	1	4	2
Maximum (ft)	62	12	60	6
Average (ft)	28	6	23	4

Notes:

1. Represents 76% of all drilled piers.
2. Represents 24 % of drilled piers.
3. Approximate values for individual drilled piers

Foundations for Parking Garage. Based on the deep rock encountered along the northern portion of the structure and the phyllite encountered at 40% of the borings, footings on rock, a soil exchange, and/or drilled piers were not feasible at this structure. We recommended that this structure be supported on footings in conjunction with compaction grouting at all footing locations to the top of rock or a maximum depth of 50 feet below footing bottom.

Foundations for Building C. Based on the proximity of the rock surface, footings situated directly on the rock and the soil exchange schemes were feasible for only the southeastern portion of the structure. The rock surface at the remainder of the building is deeper and would require the use of compaction grouting or drilled piers.

The selected foundation design for all column locations at Building C consisted of a 36-inch diameter drilled pier bearing in intact limestone or phyllite. Rock sockets were recommended for all drilled pier locations; recommended socket lengths varied in length from one to eight feet, based on the anticipated depth to rock as well as the anticipated type and quality of rock at each location. Table 3 summarizes drilled pier design recommendations for Building C.

Table 3. Summary of Drilled Pier Design Recommendations for Building C

Parameter	Minumum	Maximum	Average
Total Length (ft) *	3	124	30
Socket Length (ft)	1	8	4

* Approximate length for individual piers.

CONSTRUCTION PHASE

At the onset of construction and earthwork, solution activity became apparent. As overburden soils were removed, underlying subgrade soils and rock were exposed to the elements and sinkholes began to occur throughout the site. The variable nature of the rock and some unexpected zones of soft and loose soils also added to the complexity of the work. A subsurface cavern tall enough to stand up in was also discovered directly below one of the building foundations. Our firm was retained by the owner to conduct earthwork and foundation construction inspection activities and our field technicians made daily inspections of the site and maintained a separate "sinkhole log" to document and record repairs, in addition to their other responsibilities. During construction work, over 20 sinkholes were documented and repaired. Repairs were conducted in several different manners, including excavation and replacement with high slump grout and flowable fill, geotextile fabric and stone layering, and compaction grouting. The occurrence of sinkholes strengthened and added justification to the foundation design approach and required several adjustments to the overall program as described below. Continual interaction between the Owner, Construction Management Firm, Geotechnical Engineer, Civil Engineer and the Contractors was crucial to the success of this project. This interaction also helped the Owner re-evaluate cost-benefit issues related to construction schedule, budget, and site stability.
As stated earlier, the foundation systems originally proposed for each building were as follows:

- Building A – Spread Footings/Compaction Grouting
- Building B – Drilled Piers
- Building C – Drilled Piers
- Garage Building – Spread Footings/Compaction Grouting

The following paragraphs describe the major design and construction issues that occurred at each building location, and the engineering solutions that were implemented as the work progressed.

Building A

Building A construction began first and generally proceeded without many subsoil improvement or foundation design changes. A number of small sinkholes occurred near the building pad during construction as a result of poor site drainage, which began in the early spring.

A typical grouting layout consists of a series of primary, secondary, tertiary, and if necessary a fourth series of grout points over the treatment area. Grouting operations began after the building pad was constructed and a layer of stone was placed over the pad as a working surface. Typically, the compaction grouting operations were conducted in a grid pattern over an area that generally extended over the spread footing location and approximately 10 percent beyond the footing limits. Figure 2, located on the following page, presents a typical grouting layout.

The initial compaction grouting criteria included several conditions, at least one of which had to be met before the stages of the ground improvement process could be advanced. The criteria were as follows:

- Refusal pressure of 700 pounds per square inch (psi), or
- Primary grout pressure of 300 psi, secondary point pressure of 350 psi, tertiary point pressure of 400 psi, or
- Heave of more than 0.25 inches at the surface.

It should be noted that no initial criteria for grout volume were established.

The zone of grout improvement generally started from the top of bedrock, if present, and extended upward in two-foot intervals to approximately four feet below the footing bottoms. Surface heave, grout volume (also referred to as "grout take"), and pressure were constantly monitored during the grouting operations.

During the grouting program at this building location, a number of supplementary confirmation test borings were drilled within completed compaction grouted footing locations. These borings were drilled between grout points to assess the improvement to the soil column. Comparisons of the pre- and post- compaction grouting Standard Penetration Test (SPT) results generally

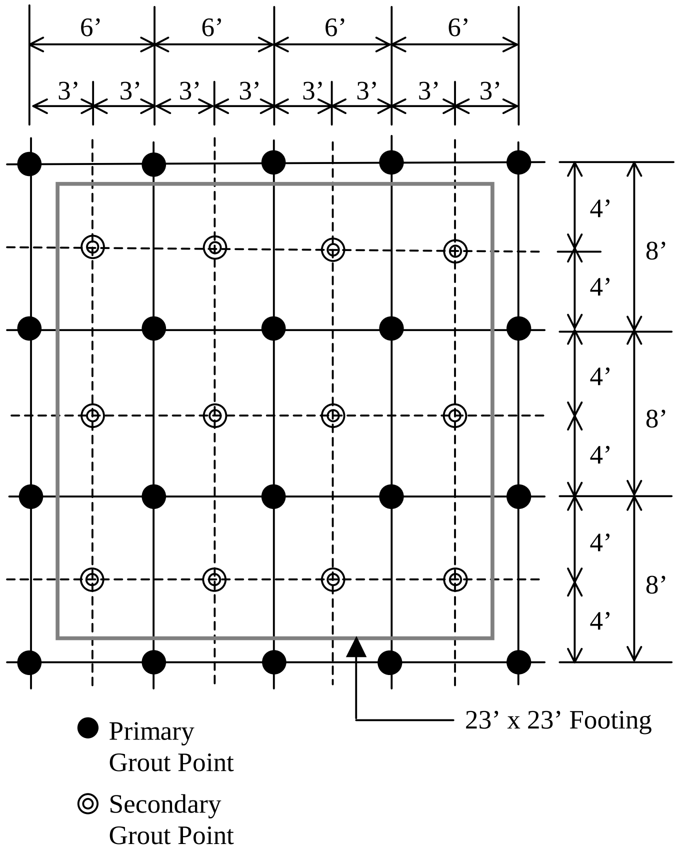

Fig. 2. Typical Compaction Grouting Layout.

revealed moderate to significant improvements. On average, the subsurface zones targeted for improvement had SPT values lower than 5 blows per foot. After the compaction grouting improvements, N-values increased to values ranging from over 10 blows to as much as 20 blows per foot, which were determined to be adequate to meet the project goals. Because the test borings could not be drilled exactly in the same locations as the grout points, the comparisons were reflective of improvements over general areas. This evaluation approach seemed to coincide well with the aerial improvements of the compaction grouting process.

Overall, the original design and actual construction activities and ground improvements at this building location were conducted within the schedule and generally within the budget. Because a grout volume condition was not established initially, grout volume take exceeded the original design estimate by approximately 50 percent. This was suspected to be attributable to the migration of the grout within the very soft soil zones. As the work on this building progressed, a grout volume criteria of 45 cubic feet per stage was established to reduce significant loss of grout. All ground improvement activities at this building were conducted as planned. Interaction of the Geotechnical Engineer, Construction Manager, Owner, and Contractor began early and this interaction became critical to the project success as work proceeded to the other buildings on this project.

Building B

The selected foundation system for this building was drilled piers founded in limestone bedrock. At the very early stages of construction, the impact of the pinnacled, fractured and uneven nature of the bedrock became apparent to the Contractor, Construction Manager and our field inspection personnel. The pier construction schedule, while aggressive, was set for completion of the drilled piers within 35 days. Figure 3 depicts a general cross-section image of the subsurface conditions typical of the limestone bedrock.

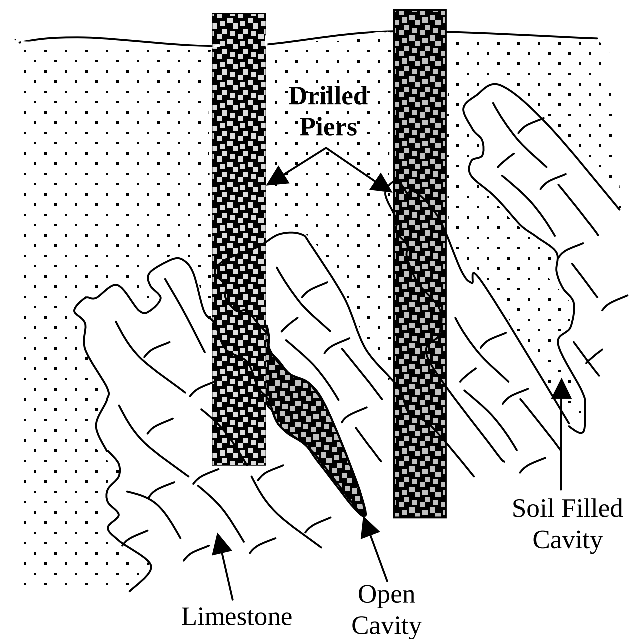

Fig. 3. Sketch of a Typical Cross-Section in Karst Terrain.

Because the conditions of the bedrock were so variable, our geotechnical project engineer was on-call throughout drilled pier construction to assist the full-time field construction crew and inspect the variable rock socket conditions and re-design socket lengths based on the integrity of the rock. In an effort to better predict the depth and competency of the rock at some pier locations, we recommended that air track drilling equipment be used. At many pier locations, a series of four to six rock probes were advanced around the perimeter of the pier location and were advanced into rock a distance of at least twice the design socket length. Additional test borings were also drilled at several locations to augment existing data and allow for correlations with the air-track probes. Using this information together with the test boring data from the geotechnical investigation, we were able to correlate and predict necessary changes in socket lengths and pier diameters. Throughout the process, we worked with the Construction Manager and Contractor to re-design some piers to allow socket lengths to be reduced and adjusted by verifying increased rock capacities and/or increasing pier diameters. At a few column locations, typically where required drilled pier lengths were minimal (i.e. total length on the order of three feet as indicated in Table 2), spread footings were designed to replace the originally proposed drilled piers.

The impact of the highly variable rock conditions could not have been accurately predicted from the results of the initial test boring program completed at this building location. The project schedule was extended from an anticipated 35 days to well over 90 days due to the delays in attaining adequate rock socket lengths and complications in maintaining alignment of the pier drilling equipment at many locations. Many times during construction, uneven and partial rock formations caused the piers to become out of alignment and this required over-reaming and time consuming coring to realign the piers. A contingency was included in the Contractor's estimate for variable conditions due to the pinnacled nature of the rock, but this was not nearly enough to cover the actual delays in the project schedule and increases in his cost to complete the project. As discussed later, these schedule and cost implications, and the Client's re-assessment of site development risks lead to foundation design changes for Building C.

<u>Parking Garage Building</u>

The foundation system selected for the garage structure included compaction grouting and continuous spread footings based on the highly variable depth to rock, zones of very soft subsoils and voids encountered within the building footprint. This building was designed as a three-level, cast in place and precast tee structure. The continuous footings were designed as quasi-grade beams that if undermined in the event of solution development, could span up to 15 foot unsupported zones. The foundation system consists basically of three continuous strip foundations that vary from approximately 10 to 14 feet wide. A design bearing capacity of 3000 pounds per square foot (psf) was used for foundation design.

The building was set into the side of an adjacent hillside and required cuts in excess of 18 feet to attain the subgrade levels. During earthwork operations, the occurrence of sinkholes and subsurface voids became quite prevalent. While excavating to attain subgrades in one foundation location, a subsurface cavern in the rock was encountered. While the rock was being removed, a subsurface cavern approximately 25 feet long and six feet high was encountered directly (less than one foot) below the foundation subgrade elevation. Our recommendation was to remove all overburden and rock to a predefined limit down to sound rock, flood the excavation with concrete, and backfill with load-bearing structural fill.

The design of the compaction grouting program was further modified to ensure coverage of areas where solution activity may be more prevalent. The initial compaction grouting design focused on areas where "very soft" to "soft" soil zones were encountered. The depth of the grouting zones varied as the depth to bedrock and soft zones varied. Because the initial test boring program did not provide for coverage at each individual column location, a series of supplementary test borings were advanced after the site was cut to subgrade level to fill data gaps. Figure 4, located on the following page, presents a typical longitudinal cross section of the building along a column line. As depicted in this figure, the depth to rock was quite variable.

At several locations within this building, the depth to bedrock was not reached within the depth of the borings and these areas were suspected as being deep solution "throats" that were choked with soft to very soft saturated fine-grained overburden soils. In these areas, we attempted a phased approach to the grouting program in attempts to confine then improve these soft, deep-seated zones. Generally, the approach was to essentially "curtain off" the deep zones with a compaction grouting ring and cap above the zone, in a manner similar to the on-going grouting program as described for Building A. The theory was to confine these areas and then grout within the confined zone to displace and improve the zone. While the practice seemed sound and the implementation proceeded without incident, there was no time or budget to perform confirmation test borings to monitor ground improvements.

Because the scope and associated cost of the compaction grouting operation at this building was significantly larger than that for Building A, our geotechnical group continually evaluated the results of the construction operations to attempt to be more cost effective while technically meeting the project needs. In a compaction grouting operation, grout volume can have a significant impact on the project costs, either to the contractor or to the owner depending on how the work is bid and contracted. Based on continuous development of the on-going grouting operations, we recommended several "test strips" along the footing location to evaluate different grouting pressures. By lowering the grout pressures, there could be an opportunity for further reducing grout take and still meeting the required soil improvement. Two test strips were set up with lower grout pressures at locations where known test boring data existed. After the grouting was completed using different pressure scenarios, confirmation test borings were drilled within the grout point grids. SPT results revealed significant increases (>20 blows per foot) in soil consistency/density that allowed us to recommend further adjustments to the grouting pressures and subsequently reduce grout take. Our revised grouting criteria based on the field testing were as follows:

- Grout take of 30 cubic feet of grout per treatment interval, or
- Primary point pressure of 200 psi, secondary point pressure of 250 psi, tertiary point pressure of 300 psi, or
- Surface heave of more than 0.25 inches at the surface.

We estimated the grout take may have been reduced by 10 to 15 percent based on this field evaluation. This field test was instrumental in later foundation system evaluations for Building C.

Throughout the earthwork, compaction grouting, and foundation construction phases at this building location, numerous solution openings occurred due to poor site drainage and leaking stormwater pipes. Some sinkholes opened up directly below the

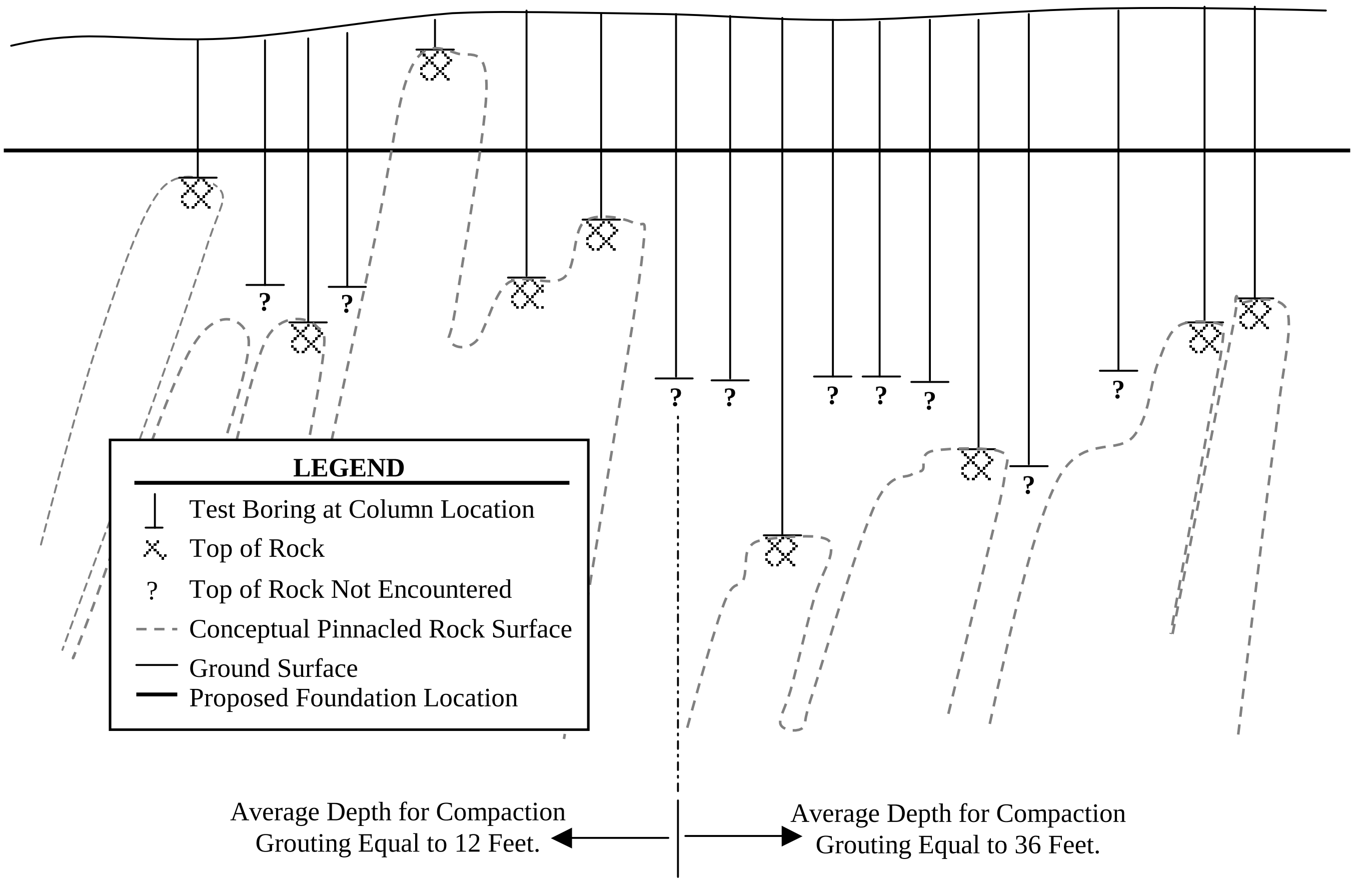

Fig. 4. Longitudinal Cross-Section of Subsurface Conditions Along One Column Line of the Parking Structure.

new foundations. Our geotechnical project engineer made numerous visits to inspect, document, and make recommendations for repair of these solution features. Where rock was exposed, the repairs generally consisted of placement of cement grout to plug the throat. Several times the grout placement was conducted in stages so that a plug could form and reduce the amount of grout necessary to backfill the void.

Again, the interaction of the Geotechnical Engineer, the field staff, the Construction Manager, and the Client were essential in establishing the compaction grouting program, solution opening repairs, and working through changes and modifications to the foundation construction and earthwork program as the work progressed.

Building C

As discussed earlier, the recommended foundation system for this structure was drilled piers. This was based on results of the the original geotechnical evaluation as well as client concerns regarding perceived uncontrollable costs associated with the compaction grouting process, the fact that the compaction grouting/spread footing alternate is more susceptible to loss of support due to solution activity and that the grouting/spread footing alternate will undergo some minor settlements. However, because the actual construction schedule for the adjacent Building B had been extended well over the contract schedule due to drilled pier construction delays, the Client and Construction Manager expressed interest in alternative foundation systems. We indicated that the spread footing/compaction grouting alternate will entail a higher risk compared to the drilled pier alternate. Also, material supplies for the proposed building, specifically the steel fabrication for the building, dictated that a faster foundation construction method be implemented.

Because the compaction grouting/spread footing approach had been implemented successfully and refined as a result of our recommended field tests, we evaluated the data needs required to fully evaluate this alternative for Building C. Based on the geotechnical data completed to date for this building, we decided that a series of supplemental test borings and several percussion probes would be required to provide data at each building column. Based on the results of our additional test borings, we concluded that a modified compaction grouting program could be implemented with a spread footing foundation alternate. Inherent with this change in the foundation system design was an understanding by the Client that additional risk from impacts by solution activity would be associated with this approach.

The implementation of the grouting program was conducted at approximately 50 percent of the spread footing locations at this structure using the modified grouting criteria as described for the Parking Garage structure and a similar phased grout point approach. The work was generally conducted within the predicted timeframe and budget at this location.

SUMMARY

Interaction of the Geotechnical Engineer, Construction Manager, Owner, and Contractor began early in the construction phase of this project and became critical to the project success as work progressed. Since our firm has had extensive experience investigating and observing foundation and earthwork activities on numerous sites underlain by solution prone limestone, we were not particularly surprised that design changes and adjustments to the foundation systems were implemented for this project. Nor were we surprised that solution openings occurred during construction. Nonetheless, as the project Geotechnical Engineer, we were faced with almost daily calls from our field inspection personnel, the Construction Manager and the Client to evaluate subsurface conditions, proposed adjustments, and design modifications. While the foundation systems were eventually successfully completed, this required substantial time and effort from our geotechnical staff.

This project presented a number of challenges that any owner, contractor, builder and engineer should be cognizant of prior to undertaking construction on a site underlain by solution prone limestone. Following is a general summary of a number of issues that were encountered while working on this project and our recommendations for managing and minimizing the impacts of these issues:

1. *Too often geotechnical investigations are considered checklist items that have very limited budget and/or are awarded to the lowest bidder rather than the most qualified candidate*. Conduct a thorough geotechnical evaluation. Also, when considering a building site in solution-prone or karst areas, select a geotechnical engineer that has extensive experience in investigating and making practical and effective recommendations for site investigation methods, foundation design and earthwork construction in limestone areas.

2. *Typically, the need for consultation with the design engineer and the need for additional subsurface data arises during construction*. If possible, consider using a geotechnical consultant that has worked on earthwork and foundation construction projects in limestone areas and has an in-house staff of construction inspectors, drillers, laboratory technicians, and geotechnical engineers who will be involved with the project from the early investigation stages to foundation design and through earthwork/foundation construction. This provides continuity on the project and, as a result, can be a critical factor in minimizing construction delays.

3. *Drilled piers are usually implemented where the rock surface is very irregular and, as a result, their lengths are often variable across a site*. If considering a drilled pier foundation alternate in limestone, solicit bids from contractors with experience working in limestone formations. Also, if possible and depending on foundation loading, use of an air-rotary downhole hammer to advance the piers into bedrock can save significant time and costs over traditional rotary methods. Limitations on the diameter of the downhole hammer (about 30 inches) will limit the use of this alternate.

4. *Solution features and uneven bedrock conditions are often encountered while construction operations are underway*. Be prepared for changes. A thorough geotechnical investigation can minimize some surprises, but the experience working in limestone areas generally indicates that a slightly larger than normal contingency for foundation and earthwork operations should be considered at the outset of construction.

5. *Construction delays resulting from difficulties associated with building in solution prone areas may result in a project running over budget and/or behind schedule*. Be flexible when necessary but stay firm on basics. Due to the fact that this project began to run over budget and behind schedule, the Construction Manager placed significant pressure on the Geotechnical Engineer to consider alternate foundation systems as well as adjustments to the designed foundation systems. While we considered and implemented changes to the grouting program and revised our foundation recommendations for one building, the on-going occurrence of solution openings during construction and even after construction was complete, emphasized the fact that there is an inherent risk involved with building on sites underlain by solution prone limestone. All involved parties should understand that each foundation system alternate has an associated level of risk for potential impacts from subsidence and should agree on the level of risk that is acceptable for the proposed construction.

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