Foundation problems in carbonate rocks of the Lehigh Valley, Pennsylvania.

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FOUNDATION PROBLEMS IN CARBONATE ROCKS
OF THE LEHIGH VALLEY, PENNSYLVANIA

by

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ABSTRACT

Much of today's urban growth in Eastern Pennsylvania is occurring in areas underlain by carbonate rocks. While limestones and dolomites provide many features conductive to urbanization, they also pose many problems for the foundation engineer. Sinkholes, caverns, irregular rock surfaces and fluctuating groundwater tables are the major causes of concern. In order to understand these problems, one must be familiar with the physical and chemical properties of carbonate rock. Then, armed with a thorough understanding of the regional geology, a suitable subsurface exploration program may be devised in anticipation of the many problems which may be encountered in the field. With a knowledge of the geology and first-hand experience derived from past projects utilizing various foundation systems in the Lehigh Valley, the geotechnical engineer may begin to develop solutions to his problems. Hence, case studies of caisson, pile and spread foundations are presented and, based on Lehigh Valley experience, a design table illustrating advantages and disadvantages of various foundations in karst regions has been formulated.
1. INTRODUCTION

Some qualities of carbonate rock are known to most people. For example, everyone knows about the spectacular caves which are often formed in limestones and dolomites. Many are also familiar with the formation of such structures and related features such as sinkholes and the subsequent topography termed karst by geomorphologists. Even the commercial values of limestones have been recognized; these include dependable water supplies from carbonate aquifers, mineable limestone for cement, building stone, aggregates and chemical processes. In addition, the residual soil left as a product of limestone weathering in humid regions provides rich agricultural soil in a region where water is abundant. With all these economic advantages of limestone, it is no wonder that these regions have become attractive areas for human habitation.

The map pictured in Fig. 19 displays the major carbonate areas of eastern Pennsylvania. It can be seen that much of the urban regions of southeastern Pennsylvania lie in limestone terrain. The low relief valleys typical of soluble rocks in humid climates comprise the Great Valley and the Lancaster Plains. The limestone areas in the York Valley southwest of Lancaster and the Chester Valley to the east are also significant.
Urbanization in these regions with many important economic advantages is producing an increasing number of costly geotechnical problems related to the same solutional features that have produced the caverns, springs, and fine agricultural soil (Knight, 1975). Whereas many people recognize the role of carbonate solubility in caves and sinkholes, few seem to relate this knowledge to the difficulties encountered by the geotechnical engineer in these areas. Since these features are usually out-of-sight beneath soil cover, they are often neglected in the planning and design stage of development. The results can often be costly and dangerous.

It is the purpose of this paper to enlighten the geotechnical engineer as to the problems which may develop in limestone regions along with some of the solutions which have been formulated. The physical and chemical properties of carbonate rock will be presented along with the mechanics of solutional erosion. Various subsurface exploration techniques will then be discussed. Finally, case studies of caisson, pile and spread foundations will be presented along with a design table formulated through Lehigh Valley experience.
2. ENGINEERING PROPERTIES OF CARBONATE ROCK

The engineering properties of carbonate rock may best be depicted after a basic understanding of limestone deposition and consequent development of karst topography is explained. "Karst includes the circulation of water in fractures, fissures, joints, and other cavities, and the development of corresponding forms in soluble, mostly carbonate rocks". (Herak, et al., 1972) The process and the resulting landforms may reach to great depths in rock complexes, but often the ground surface hides the great irregularities that may exist underground. It is these irregularities and the constant alteration of the carbonate bedrock which creates the major problems for the geotechnical engineer.

Geologists believe that most carbonate rock forms in shallow marine waters where there exist abundant plant and animal life. Disregarding details and exceptions, (since there exist varying opinions and insufficient supporting evidence) Limestones form by the precipitation of calcium carbonate (CaCO₃) from solution through the action of plant and animal organisms, and in some rare cases through chemical reactions themselves. Some organisms extract CaCO₃ from the water in order to form bony skeletal structures or shells, while others emit substances resulting in the precipitation of soluble constituents. Most of this material accumulates near land as ooze,
lime muds, or shell bank deposits which often contain particles of silt, sand, or shell fragments which act as nuclei for oolites. These are small nodules of spherical precipitate layers of carbonate material surrounding the foreign particles. The large amount of pore space existing between grains and fragments of these freshly deposited formations account for their high void ratios. Shell bank deposits (coquina) typically display void ratios of 1.0 - 1.5 while lime muds and ooze with their smaller particle sizes have void ratios of 0.5 - 1.0 (Sowers, 1975). These properties, however, change drastically as the deposits are subject to lithification.

Consolidation of these deposits by subsequent deposition and cementation of particles due to additional precipitation of carbonate material are the two major mechanisms by which limestones are lithified. Both of these processes reduce void sizes and corresponding void ratios drastically. During these processes, chemical replacement of more soluble calcium ions with less soluble magnesium ions might occur during induration forming dolomite.

If this process is allowed to continue, the results are well indurated limestones having a void ratios of less than 0.1 and compressive strengths of 70 000 kN/m$^2$ - 150 000 kN/m$^2$ (10,000 - 20,000 psi) (Sowers, 1975). The average engineering properties of carbonate rock are displayed in Fig. 1.
3. SOLUTION OF CARBONATE ROCK

3.1 Chemical Properties

Approximately sixty minerals occur in nature that have the CO$_3$ group in common. Of these, calcium carbonate is the most predominant in modern sediments. However, in ancient rocks, calcite and dolomite CaMg (CO$_3$)$_2$ are by far the most common, comprising nearly 100% of the carbonate minerals in carbonate rocks (Blatt, et al, 1972). The main characteristic of importance in this discussion, however, is the fact that all of these carbonate rocks are subject to solution in a weakly acidic environment. The formation of theoretical carbonic acid from water and carbon dioxide described in the equation below creates a potent solvent for the solution of carbonate rock.

$$3\text{CO}_2 + 3\text{H}_2\text{O} \rightleftharpoons \text{H}_2\text{CO}_3 + 3\text{H}^+ + \text{CO}_3^- = \text{HCO}_3^-$$

Organic decay also contributes to the total acidity of water. The following equation displays the solution of dolomite when subject to the acid produced in this acidic environment.

$$\text{CaMg (CO}_3\text{)} + 2\text{CO}_2 + 2\text{H}_2 \rightleftharpoons \text{Ca (H CO}_3\text{)} + \text{Mg (H CO}_3\text{)}$$

It is this type of chemical reaction that can bring about drastic changes in the bedrock in a karst region.
The solutional characteristics of any carbonate rock are a function of four major variables: (1) the physical and chemical characteristics of the groundwater, (2) the chemical and lithological characteristics of the rock, (3) the initial distribution of pore space in the rock, and (4) the rate of groundwater flow expressed in terms of yield per unit area. A basic understanding of these variables is necessary for the geotechnical engineer to make any assumptions as to the rate and effects of solution on the bedrock.

The physical and chemical characteristics of both the groundwater and the bedrock act almost synonymously. They define the maximum intensity at which the solutional process may take place. For example, strongly acidic waters formed by the leaching of mineral rich soil by percolating rainfall will react actively with a strata of calcite rich bedrock. However, the actual intensity of the solutional process is also a function of the surface area of carbonate rock exposed to groundwater. Thus, the importance of the lithological structure (cracks, fissures, joints, bedding planes, etc.) along with the amount and distribution of pore space in the rock becomes apparent. It is through these passages that the groundwater may flow and actively attack the carbonate rock; however, without a reasonable flow of groundwater through the carbonate aquifer, none of these mechanisms would have the capacity to dissolve any of the bedrock.

It now becomes obvious that the amount and direction of groundwater flow is the dominate factor in determining the amount and rate of solution governing karstic development. Variations in lithology
become secondary in nature, since the solutional process may only continue with the fresh supply of acidic water flowing through the carbonate rock. This fact denotes the importance of hydrology in karstic development.

3.2 Hydrological Properties

The hydrology of a carbonate aquifer is simply a study of the groundwater flow through a particular region. This property governs the formation of underground caverns, channels, and related formations such as sinkholes. The theories as to the development of these underground drainage systems will now be presented.

Some geologists (Mandel, 1967) believe that the circulation of water in a karstic region is restricted to underground streams flowing more or less similar to surface streams. These regional systems of solution channels develop only if the general hydrological conditions (especially the location of base level) remain constant during long time intervals. Only then will preferential directions of flow develop indicating the oldest and most stable environmental conditions. The reader is directed to the work of Gardner and Gray (1975) in which a method of tracing subsurface flow using artificially colored spores is outlined.

In the saturated zone of carbonate aquifers, erosion by solution usually begins to take place in the following manner. First, solution channels form and are widened near springs where a large flow occurs. Channels are widened until one "giant" spring captures the entire flow of the region. Then, an intricate system of solution
has developed having a similar direction of outlet. This indicates a
general change over a period of time from an isotropic to a strongly
anisotropic aquifer.

As for the relative rates at which solution dissolves carbonate
rock, there have been two basic arguments. Meinzer (1923) argued that
limestone dissolved most rapidly above the groundwater table where
there exists abundant and rapid percolation of rain water rich in the
CO₂ necessary for solution to take place. Davis (1930) refuted this
argument by stating that rapid solution resulting in cavern formation
occurred most rapidly below the groundwater table. He believed that
the deep circulation of groundwater in the phreatic zone was the
reason for this observed phenomena. Support for this theory came
twelve years later in work done by Bretz (1942). One fact has been
agreed upon, however. The top of the groundwater table is most sus-
ceptible to solution by the lateral movement of groundwater, while
the zone of aeration above the groundwater table is more prone to so-
lution by downward water movement. In either case, it becomes
evident that carbonate rock is a dynamic material. It is in an ever-
changing state of flux with its environment and thus drastic subsur-
face changes may occur. Depending on many variables effecting so-
lution, these changes may occur more rapidly than most events on a
geologic timescale. Often discontinuities develop within the design
lifespan of a structure, or possibly in the construction period. It
therefore becomes imperative for the geotechnical engineer to be
aware of any such possible changes in order to correctly design
foundations in such an everchanging environment.
4. KARSTIC EROSION AND SINKHOLE FORMATION

Karstic erosion is defined as essentially the solution of calcareous rock by water; with the importance of the solutional and hydrological characteristics of carbonate rock in mind, one may begin to understand the mechanics of karstic erosion and the formation of sinkholes.

Solution of calcareous rock enlarges the pore space in the rock and thus increases its overall void ratio. The result of this process is greater porosity which: (1) enhances water circulation and thus promotes further solution, and (2) decreases overall strength since stress is distributed within the remaining rock skeleton. The flow of water through these pores is the agent for solutional erosion.

Sowers (1975) noted that above the groundwater table, flow is predominantly downward which causes the formation of solution pits. Below the water table, the predominant horizontal movement of water creates irregular conduits or small caves. The growth of these channels concentrates flow and thus increases the rate of solution making this entire phenomena a self-aggravating process.

The solutional process itself is most intense at the bonding points of the rock skeleton according to Sowers (1975). The solutional activity destroys the induration of the bedrock thus reducing its strength and leaving the insoluble constituents as residual soils.
It is this dynamic, everchanging character of limestone which makes it prone to engineering problems.

Sinkholes are the specialized form of karstic erosion which are of special interest to the geotechnical engineer. They form in regions where less soluble overlying deposits containing resistant erosional beds exist. Solution escarpments develop leading to the formation of vertical shafts and ultimately, sinkholes. There are two major forms of sinkholes which are of interest, funnel and collapse sinkholes.

Collapse sinkholes form in regions where the underlying solution prone limestone is covered by a competent, non-solution prone stratum. Shale, sandstone or even residual soil may provide the necessary strength to promote the development of a collapse sinkhole. Large caverns form in the underlying limestone as a result of concentrated solution and/or high solubility. The resistant overlying layer acts as a bridge permitting the cavern to grow until the overburden pressure causes the cavity to collapse. Figures 2 and 3 illustrate a collapse sinkhole both schematically and photographically.

According to Sowers (1975) "a number of factors may initiate failure, but increased solutinal activity is ultimately responsible". Among the reasons for this increased activity are chemical changes in the water, increased flow due to heavy rainfall, pipe leaking, or pumping of groundwater. Increased surface loading may also help to initiate a failure which always occurs suddenly. Figure 3 from Garner (1974) is an aerial view of a collapse sinkhole in Shelby.
County, Alabama whose failure was believed to have been caused by the lowering of the groundwater table and the corresponding increase in effective stress in the soil. The depression measures 130 meters (425 ft.) long, 110 meters (350 ft.) wide, and 45 meters (150 ft.) deep. Fortunately, large scale dropouts such as these are not common and can often be located at a project site before construction begins.

Funnel sinkholes, on the other hand, are the most dangerous of all subsidence phenomena associated with carbonate rock. They may develop suddenly without notice and are difficult to locate before they pose a threat. They are also the predominant sinkhole form in Eastern Pennsylvania.

A funnel sinkhole forms when a solution-enlarged opening extends upward to the soil bedrock interface. According to Sowers (1975), as these solution cavities grow they form an interconnected channel of slots, chimney-like holes and joints which permit seepage and groundwater to erode away the overlying soil. Figure 4 illustrates the sequence of events leading to the formation of a funnel sinkhole, while Fig. 5 displays a cross section of an actual funnel sinkhole.

At the outset of this subsurface erosional process, the overlying soil bridges or arches over the solution cavities. As time passes, soil begins to erode away into the solution channels by various mechanisms. Sowers (1975) describes the peeling off of thin slabs of soil from the roof of the arch. This phenomena termed "roofing" is caused by the overstress in the internal shell of the dome resulting from changes in moisture and surface softening due to
percolation of water. Thus this process acts most rapidly during periods of high rainfall and infiltration.

In order for this process to perpetuate itself, there must be enough groundwater flow and large enough cavities in the rock to permit the transportation of the eroded soil particles. If the eroded soil is not carried away, it soon clogs the cracks and pores in the limestone and temporarily puts an end to the process.

As the roofing and seepage erosion processes continue, the cavity above the bedrock enlarges. In cohesive soils, the cavity is domelike in contrast to the narrow chimneylike cavities developed in silts and sands. As these cavities grow, the overlying soil may no longer bridge the solution pits, and subsidence begins. This subsidence can be a rapid dropout of a soil plug or a slow settlement phenomena depending on the properties of the soil and the size and location of the solutional fissures. Figures 6 and 7 illustrate the typical surface depression resulting from subsidence into a funnel sinkhole.

Sowers (1975) points out that there are a number of factors which accelerate the development of cavities in the soil and the final dropout of the soil plug. As mentioned before, rapid moisture changes accelerate the roofing phenomena. The lowering of the groundwater table is also a major aggravating factor effecting the rate of subsidence. This has the effect of: (1) increasing downward seepage gradient thus increasing downward erosion; (2) reducing capillary tension in cohesionless sands and thus increasing its ability to flow.
through narrow passages; and (3) causing shrinkage cracks in plastic clays which weakens the mass in dry weather and produces concentrated seepage paths during rains. An increase in infiltration, especially after a prolonged dry spell often initiates a failure; this is especially true when the rainfall accompanies a previous drop in the groundwater table.

From the previous discussion of the mechanics of sinkhole formation, it becomes obvious that the occurrences explained may pose serious problems for the geotechnical engineer. It follows that accurate subsurface investigation techniques must be utilized in order to discover the actual condition of underlying carbonate bedrock. The location of sinkholes and/or the prediction of possible sinkhole locations is necessary in order to properly design the foundation for any structure. Many methods of subsurface exploration are in use today, each having its advantages and disadvantages. It becomes the duty of the foundation engineer to devise the most suitable exploration survey for sites underlain by carbonate bedrock.
The proper design of any civil engineering structure requires an adequate knowledge of subsurface conditions. This is especially true in karst regions where conditions may vary extensively within a few feet as in a pinnacled limestone region, or may change due to solutional erosion in the mere time span of construction. Precise data pertaining to both the existing and the projected structure and properties of the carbonate substrata is of the utmost importance. It is for these reasons that extensive subsurface exploration programs are recommended for any structure to be founded on carbonate rock.

There are a number of methods for exploring subsurface conditions in karst regions today. Occasionally, one method is found sufficient for a particular case; however, a combination of these techniques is often the most satisfactory answer. Of the exploration methods in use today, the use of boring and excavation techniques, geophysical sounding, gravity, radar, air photos and electrical resistivity will be discussed.

5.1 Aerial Photographs

Aerial photographs have become one of the most useful sources of information for any site investigation. When used in conjunction
with topographic and geologic maps, this tool becomes especially useful in karst regions. In the hands of a specialist, air photos may be used to locate regional jointing patterns along with other important geologic structures. These features serve to enlighten the engineer as to the general amount and direction of groundwater movement so instrumental in karstic erosion. Sinkholes themselves may often be located from photos. Figure 8 displays a regional jointing pattern and associated sinkholes formed by solutional activity in a karst region.

Although air photos provide much information, they are basically reconnaissance tools for use on preliminary large site selections. Information obtained from aerial photography must be verified and expanded by actual site investigation techniques.

5.2 Geophysical Explorations

Geophysical exploration techniques have proven to be useful as a rapid means of obtaining economical supplements to borings in preliminary exploration surveys. Information pertaining to rock profiles, relative densities, discontinuities, and location of groundwater table comprise the core of basic information sought by the geotechnical engineer in a karst region. In the hands of one experienced in both soil and geophysical theories, data from these surveys may be interpreted to fine detail requiring sometimes only spot checking by borings and/or other direct means of exploration. The role of borings, however, may not be overstressed. Figure 9 provides
a table which evaluates the advantages and disadvantages of each geophysical technique in karst regions.

5.2.1 Seismic Refraction

Seismic surveys fall in two categories; reflection and refraction. Seismic reflection techniques have been used by the petroleum industry to investigate strata at depths of over 300 meters (1000 ft.) and hence are of little use to the geotechnical engineer in a karst region. Refraction methods, however, are effective at depths up to 300 meters (1000 ft.) and therefore of most use as a subsurface investigation tool in a carbonate rock region.

Both of these methods however, are based on the same principle. Shock waves travel at different velocities through different types of materials. The velocity of propagation is a function of many variables such as density, texture, moisture content, void space, and elastic properties. Since these properties should vary between materials comprising different strata, the nature of the material along with its stratification characteristics may be theoretically determined. (Dobrin, 1960)

In any seismic exploration, shock impulses are imparted to the ground by means of explosives, mechanical impact or mechanical vibration. These impulses generate compressive, shear, and surface waves of which the longitudinal compression waves are of most importance. The waves traveling away from the source are classified as direct, reflected or refracted waves. Direct waves travel in a
straight line from the source to the surface, while reflected waves are those which are turned back when they encounter a media possessing a different seismic velocity. Of importance to geotechnical engineering purposes, however, are the refracted waves. These waves undergo a change in direction of propagation when they encounter a media of different seismic velocity. It is this property which forms the basis of all seismic refraction surveys.

Figure 10 (Lowe and Zaccheo, 1975) displays a typical arrangement of equipment for a field investigation. Point A is the source of the seismic impulse while points $D_1 - D_{12}$ are detectors (geophones) which record the first arrival of shock impulses. The degree of detail to be obtained from a survey is a function of the geophone spacing and in general, the distance from $D_1 - D_{12}$ should be three to four times the depth to be investigated.

Data obtained from the "first arrival times" at each geophone is plotted on a corresponding time distance graph shown at the top of Fig. 10. The slope of each segment represents the inverse of the characteristic velocity of its corresponding conducting medium. The distance $X_c$ represents the critical depth at which there is a change in medium.

Formulas developed for determining strata thicknesses and dip are outlined in Dobrin (1960). These formulas are based on the following assumptions however:

1. Each stratum is homogeneous and isotropic
2. Strata interfaces are planar
3. Each stratum is of sufficient thickness to reflect a change of velocity on a time-distance plot
4. Characteristic wave velocity for each stratum increases with depth

(After Lowe & Zaccheo, 1975)

Often times, all these requirements are not satisfied. It is then the task of the engineer to interpret the data based on experience, a number of boring logs, and engineering judgment. For example, a stratum with a thickness less than one-fourth of the depth from the ground surface to the top of the stratum will not be indicated in a time versus distance plot. (Lowe & Zaccheo, 1975) The same is true for the condition where a stratum of low velocity underlies a high velocity. The existence of such a layer is "masked" due to the downward propagation of refracted rays. In addition to these problems, the characteristic velocities for various materials may overlap and thus make it difficult to distinguish the interface between two strata.

Special techniques and corresponding equations are used for locating discontinuities and dipping strata. These are all outlined in Dobrin (1960). It is a standard practice "reverse profile" in order to determine the dip of strata. If this is not done, a single time distance plot would simply indicate an average stratum thickness for horizontal beds, see Fig. 11. As for subsurface discontinuities,
empirical formulas are used to decipher such features as weathered strata, faults, and subsurface channels. Of most use in the location of sinkholes is a technique called fan shooting.

If a large discontinuity believed to be a sinkhole is discovered, a typical arrangement of field equipment pictured in Fig. 12 is made. Normal refraction times are recorded by the detectors at the edge of the fan radius while the time difference or "lead" is recorded by those detectors which receive the impulses traveling through the sinkhole area. The difference in arrival times for the two paths is due to the inherent low velocity of the air or soil-air plug that fills the sinkhole. With a second fan shot at approximately right angles to the first, one may then obtain a fairly reasonable outline of the sinkhole.

Refraction profile surveys are also utilized in order to deduce the subsurface geometry in an area. Successive shots are taken at uniform intervals along each line, and successive detector spreads are shifted about the same distance as the corresponding shot points so as to keep the range of shot-detector distance the same for all shots. Data from such a survey may be used in conjunction with boring logs in order to plot the rock contours in a limestone region. The major problem with this procedure (Dobrin, 1960) lies in separation of intercept times into proper delay times corresponding to the respective depths of the layers under shot and receiver. Experience is the only way in which this data may be interpreted; this fact can not be overemphasized.
One may now comprehend some of the uses of seismic refraction surveys in karst regions. Its primary application lies in the reconnaissance stage of any project for which it provides a rapid, efficient means that often permits an engineer to evaluate a site without having to perform a large number of borings. Among the information such a survey might produce is: depth of bedrock, thickness of stratum, location of groundwater table and location and extent of subsurface discontinuities. All of these factors are necessary for the design and construction of any structures in a limestone environment.

5.2.2 Electrical Resistivity

The use of electrical resistivity has proven to be a rapid geophysical tool for providing subsurface information. It provides an economical supplement to boring exploration programs for geotechnical purposes. This method may be used to define limits of granular borrow areas and organic deposits, and to locate the groundwater table. In karst regions, earth resistivity may be used to supply information necessary for bedrock profiling.

Resistivity is a fundamental property of a material which characterizes that material almost as completely as its density. It is defined as the resistance in ohms between opposite faces of a unit cube of material, see Fig. 13. Field resistivity measurements afford an opportunity for distinguishing one type of material from another without having to perform an actual excavation. This is accomplished by equipment which introduces electrical currents into the ground.
at various depths and then measures the resistivity at those depths, see Figs. 14 through 16. Then, by the determination of vertical and lateral variations in resistivity, it is possible to infer the stratification and extent of subsurface deposits. This extrapolation, however, process is subject to certain limitations.

Two basic field techniques utilizing earth resistivity are presently in use. They are electrical profiling or traversing, and electrical sounding. The basic difference between each of these methods lies in the spacing of electrodes of the resistivity apparatus. Since the depth of investigation is a function of the spacing between electrodes it follows that traversing, with its constant spacing, yields a uniform depth of investigation, while sounding and its varying electrode spacing provide information varying with depth.

Electrical profiling is a technique normally utilized in a rapid survey of an area. It is particularly suited for the location of faults or fault zones, and the location of steeply dipping interfaces between different types of earth material. Thus it is the basic method utilized in karst regions. A proper electrical traverse would indicate the formation of vertical shafts which might be subject to sinkhole formation, and even locate sinkholes themselves.

The electrical profiling technique utilizes a suitable electrode spacing throughout the survey. This spacing is chosen according to the estimated depth of bedrock to be investigated. It has been found that the depth of investigation is approximately equal to the spacing between the outer electrodes of the resistivity apparatus. The
survey proceeds along the surface of the site in a grid pattern providing information as to the changes in subsurface strata lying beneath the survey and above the depth of penetration. A profiling survey thus may be considered as an "electrical trench", since it may only detect lateral variations in subsurface conditions. A typical profile survey is illustrated in Fig. 17.

The second basic field procedure is the electrical sounding technique. The purpose of this method is to provide information as to the variation of subsurface materials with depth. It is especially useful in determining the degree of weathering and soundness of carbonate rock with depth along with estimating layer thicknesses, and the location of the groundwater table. In this procedure, the center of the electrode spread remains constant while the electrode spacing is gradually varied. As the spacing is increased, the effective depth of the survey increases, making this procedure an effective means for analyzing stratification, and irregularities in bedrock. A typical sounding survey arrangement is pictured in Fig. 18.

It is clear that a combination of electrical profiling and sounding would be the best procedure to use in a karst region. Theoretically, this would provide all of the subsurface information required by the geotechnical engineer for design in a karst region. The profiling survey would provide a contour map of the underlying bedrock, while the sounding survey would yield information pertaining to the actual depth, soundness, and irregularities of the
underlying bedrock. However the fact remains that there are many limitations to the use and reliability of earth resistivity data and its interpretation. Among the factors affecting data interpretation are the broad range of resistivity values for a given material, the overlap of these ranges for different materials, near surface irregularities, and the existence of stray potentials.

Field measurements of known materials have provided ranges of resistivity values to be used in conjunction with raw data obtained from a field survey. However, these resistivity values cover a large spectrum due to variations in water content, particle size, stratification and pore space. This also creates overlaps in the characteristic resistivities of different materials. Both factors make the determination of the exact nature of the material present along with any stratification difficult or impossible for some materials. In the case of karst topography, this becomes important when the groundwater table is to be established. If clay lies above a carbonate bedrock, the water table may not be reflected in the data because of the double layers of water and ions surrounding each clay particle even above the groundwater table.

Near surface irregularities pose another threat to the interpretation of field data. Surface features such as ditches, roads, and filled depressions are boundary conditions which tend to distort the normal pattern of the induced electrical field. Among the most important surface feature is the existence of lateral variations in resistivity. This occurs when there is a vertical interface between
two materials of different resistivity. This also distorts the potential field and may cause serious error if the interpreter is unaware of the interface. It can not be stressed enough that the data interpreter must possess a large amount of experience; for this there may be no exception.

A third source of error in resistivity measurements lies in the electrical potential generated by ore bodies or stray potentials. These occurrences tend to superimpose themselves on the induced potential field and create other sources of error. Particular attention should be given to buried pipelines, electric cables, and other underground structures. These structures tend to limit the usefulness of resistivity methods in urban areas.

Keeping the limitations of earth resistivity in mind, it can be seen that the methods described may be useful as a tool aiding in the geophysical exploration of a karst region. The interpretation of the data by personnel experienced in both resistivity techniques and the geology of a given area is of course essential. This fact is particularly true of karst regions and their inherent complexities. The methods do provide a fairly accurate and economical technique when used in conjunction with reliable soil boring data.

5.2.3 Earth Penetrating Radar

A number of electrical survey techniques utilizing electromagnetic waves generated at the surface by current alternations in loops on the ground or in aircraft have been developed. These
Electromagnetic waves possess the same frequency as the alternating frequency of their origin which may vary from a few cycles up to thousands of kilocycles. Fountain (1976) describes the use of a van-mounted system generating a frequency spectrum at about 30 MHz. and 120 MHz. to investigate subsurface cavities in limestone regions.

Electromagnetic waves attenuate in the earth at a rate depending on the frequency and electrical characteristics of the earth. Waves of a higher frequency drop off in intensity more rapidly with depth. When these waves encounter a conducting formation, they induce currents in these bodies according to electromagnetic theory. These newly induced currents then become a source of waves which may be detected again at the surface. Thus, subsurface anomalies would appear as variations in electromagnetic conductivity.

Fountain (1976) notes that the theoretical depth of penetration of electromagnetic waves is a function of the frequency. An optimum frequency for greatest penetration was found (Peters and Bardeen, 1932) to vary according to the following formula:

$$ h \frac{f}{p} = 10 $$

with

- \( h \) = depth of investigation in meters
- \( f' \) = optimum frequency
- \( p \) = resistivity in ohm-centimeters

Present techniques of measuring electromagnetic waves at very low frequencies is so ineffective that the depth of penetration is limited to 500 meters (1500 ft.) according to Dobrin (1960). This
is, of course, more than adequate for any geotechnical investigation in areas where the electrical properties of the earth are within the normal range for sediments. Karst regions, however, often do not fall into this category.

Fountain (1976) outlines and evaluates the use of ground penetrating radar in limestone regions with special emphasis on the location of subsurface cavities. The results of his work show that radar is capable of detecting voids in some but not all materials depending upon their characteristic conductivities. When detected, however, echoes from both the floor and roof of the cavity provide an accurate estimate as to its size. The method outlined did fail to resolve 60 - 90 cm (2 - 3 ft.) diameter vertical pipe cavities in low loss earth material characteristic of many sinkhole environments. The method does, however, possess a high search rate and immediate data printout, although Fountain found it to be an inadequate technique for locating sinkholes and other cavities. This is basically due to its limited penetration depths in the high loss soils associated with karstic erosion.

5.2.4 Gravity

Basically, gravity surveys measure lateral variations in the earth's gravitational field that are associated with near-surface changes in density. The theory behind gravitational surveying depends directly on Newton's Law expressing the force of mutual attraction between two particles in terms of their masses and separation. This law states that the mutual attractive force (F) between
two particles of mass \( m_1 \) and \( m_2 \) separated by a distance \( r \) is evaluated by the following equation:

\[
F = G \frac{m_1 m_2}{r^2}
\]

with

\( G = \text{Universal Gravitational Constant} \)

Since mass is related to density, a low density area characteristic of a void or soil-air plug in a sinkhole would result in a drop in the gravitational field. It is this principle which is the basis for all gravity surveys in karst regions.

A gravity survey consists of a series of measurements of the relative attraction of a mass to the earth. When each measurement is corrected for elevation, topography, and location on the theoretical earth spheroid, anomalies become evident. Survey and correction techniques are outlined in Dobrin (1960). In karst regions, gravity "lows" are usually associated with cavities or sinkholes. Empirical equations exist for use in estimating the size, shape and depth of such anomalies; these are also outlined in Dobrin (1960).

Fountain (1976) evaluates the use of gravity surveys for the location of subsurface voids in karst regions. Tests proved that gravity surveys were fairly successful in locating large subsurface cavities. A large number of near surface cracks and joints acted as "lithological noise" making the recognition and interpretation of gravity anomalies more difficult. Small voids, however, were found difficult to detect with any degree of certainty, since the associated drop in gravitational field is so minute. Thus, this technique...
requires extensive verification through the use of boring logs.

5.3 Boring Methods

The major purpose of any subsurface boring program is to provide the necessary detailed information required for the design of a structure. A boring may be defined (Lowe & Zaccheo, 1975) as any vertical, inclined, or horizontal hole drilled in the ground for the primary purpose of obtaining samples of the overburden or rock materials present. The information obtained may be used to determine the stratigraphy and/or the engineering properties of those materials. The boring hole itself may also be used for determining such properties as shear strength permeability, observance of fluctuations in the groundwater table, measurement of pore pressure and the measurement of deformations.

Two basic operations are required in any boring exploration program, they are: (1) advancing the hole and (2) sampling of both soil and rock. The large variability of materials to be sampled has prompted the development of many different techniques, all of which are beyond the scope of this paper. What is important, however, is the fact that the variability of material in a karst region often requires an extensive program of borings.

The residual soils which blanket a well weathered carbonate rock have been found to vary considerably from sedimentary soils of similar classification. Also, the engineering properties of these
soils often vary laterally and with depth. In order to estimate accurately the overall properties of the soil on a particular site, an extensive boring, sampling and testing program is necessary.

The same fact holds true for the sampling of carbonate bedrock. Different weathering degrees and profiles provide a large variation in the strength properties of the rock. Joints, cracks, and solutional voids must be located and sampled in order to provide the entire picture of the subsurface conditions. Boring logs are the only positive means of determining subsurface conditions.

It may now be noted that the use of boring logs combined with one or more methods of geophysical exploration provide the most extensive subsurface information. The assumptions made during the geophysical analysis may be verified through the use of key boring logs at which the actual engineering properties of the soil and bedrock may also be analyzed. The choice of the proper methods to be utilized is left to the geotechnical engineer, but his judgment should be based on many factors such as: degree of accuracy necessary, design life of structure, cost, past experience and field limitations. It is only through a careful analysis of these factors that an inexpensive and informative survey may be devised.
6. GEOLOGIC HISTORY AND FOUNDATION PROBLEMS IN CARBONATE ROCKS OF THE LEHIGH VALLEY

The following section is a regional study of the Lehigh Valley. Its purpose is to explain the regional geology of a specific karst region and then present some of the actual geotechnical problems and their corresponding solutions. Finally, based on engineering experience obtained in the Lehigh Valley, a design table presenting the advantages and disadvantages of pile, caisson, and spread foundation is formulated.

The Lehigh Valley is the local name for a physiographic feature (Fig. 19), the Great Valley of the Appalachian Valley and Ridge Province, that extends from New Jersey to the southern Appalachians. To the west in Pennsylvania, the Great Valley is called the Lebanon Valley and west of the Susquehanna River at Harrisburg it becomes the Cumberland Valley. However, throughout its extent the Great Valley is underlain by similar rocks and consequently many, but not all, of the problems encountered during construction in the Lehigh Valley would be typical of the Great Valley as a whole. The unique feature of the Lehigh Valley with respect to the Great Valley to the southwest is that portions of the Lehigh Valley have been overridden by glaciers and the debris left behind by the melting ice has complicated the picture in those areas.

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Physiographically, the Lehigh Valley is a long, linear topographic depression, striking approximately N 70° E. It is bounded on the south by a series of relatively high hills (the Reading Prong) composed of high-grade metamorphic granulites and other crystalline rocks known locally as South Mountain. The northern boundary of the valley is marked by a prominent ridge of quartzite (Kittatinny Mountain) that persists as a topographic high throughout the length of the Appalachians. Longitudinally, the Lehigh Valley can be divided in half along a topographic break separating a slightly higher and more dissected area underlain by shale and slate, the northern half of the valley, from a lower, flatter region containing more gently rolling hills and underlain by a thick sequence of Cambro-Ordovician carbonate rocks. These limestones and dolomites have been subjected to a long history of deformation (at least two episodes) and subsequent weathering and erosion has produced in them a network of solution channels that have led to the development of sinkholes and related features (pinnacles, etc.) throughout the southern portion of the Lehigh Valley.

It is the purpose of this paper to present the general geologic setting, development, distribution and characteristics of sinkholes within the rocks of the Lehigh Valley, and the engineering practices dictated by their occurrence.
Rocks in the southern portion of the Lehigh Valley consist of a series of Orthoquartzite-carbonate shelf deposits laid down during the early "geosynclinal phase" of the Appalachians. They lie along the northern side of the Reading Prong and also crop out in intermontane valleys within the Prong. The Saucon Valley and Oley Valley would be local examples of these intermontane valleys. These rocks were intensely deformed during the Ordovician Taconic Orogeny. Later periods of deformation during the Paleozoic Era also influenced their structural development. In the northern half of the Lehigh Valley, the Cambro-Ordovician carbonate rocks are overlain by a great thickness (3660 meters or 12,000 ft.) of eugeosynclinal turbidites known as the Martinsburg formation. Since these rocks are not prone to sinkhole development, they will not be discussed further.

Stratigraphic units of importance to this discussion are shown in Fig. 20. As can be seen from this table, the lower portion of the stratigraphic column contains approximately 1525 meters (5000 ft.) of carbonate rocks. These are the rocks that underlie the southern half of the Lehigh Valley. Not all of these rocks are equally subject to erosion of the type that might lead to cavern or sinkhole development. However, because of the structural complexity within the area, distribution of solution prone units can occur anywhere within the outcrop belt of carbonate rocks.
The rocks have been thrown into a complex pattern of large recumbent folds or nappe structures in which the tectonic transport to the northwest has been considerable.

According to Drake (1970) the Precambrian Rocks of South Mountain were also involved in this deformation and represent the crystalline core of a large nappe structure. The Precambrian rocks, according to this model are unrooted and are underlain by the same carbonate rocks that comprise the Great Valley sequence. Therefore, sinkhole development might be expected anywhere where these carbonate rocks are exposed within the Reading Prong, as well as in the Great Valley Proper.

Later periods of deformation have superimposed a series of more open folds and brittle fractures (faults and joints) on these rocks and since the Mesozoic they have been subjected to a long period of erosion and solution. Much of the solution of carbonate rocks has proceeded along joint or fault planes and bedding planes have exerted relatively little influence on the development of sinkholes.

Much of the Lehigh Valley in the vicinity of Bethlehem and Allentown was later subjected to the advance of an Illinoian ice sheet which left a veneer of glacial drift over the valley floor. This glacial material filled many existing sinkholes and masked any evidence of developing karst topography that was present at that time. Consequently, portions of the valley lack even the subtle evidence of sinkholes that can be found in unglaciated areas.
6.1.1  **Stratigraphy and Weathering Characteristics of the Carbonate Rocks**

Initial Cambrian sedimentation laid down approximately 30 meters (100 ft.) of clastic rocks, the Hardyston formation, made up of arkose, feldspathic sandstones, and quartz pebble conglomerates which grade into the first carbonate rocks of the valley sequence, the Leithsville formation.

The Leithsville formation consists of an interbedded series of light to dark gray dolomite and tan phyllite with small stringers of quartz sand. It is estimated to be on the order of 300 meters (1000 ft.) thick. Contacts with the underlying Hardyston formation and overlying Allentown formation are gradational. The Leithsville formation weathers quite readily and therefore is poorly exposed throughout the valley. The susceptibility of this unit to weathering also makes this unit a prime one in which to develop solution channels.

The Allentown formation, also Cambrian in age, overlies the Leithsville and is made up of 520 meters (1700 ft.) of characteristically alternating light and dark gray, thin to massively bedded dolomite, oolitic or sandy dolomite, and minor shaly dolomite. Stromatolites are common in this unit as are ripple marks, crossbeds, mudcracks, edgewise conglomerates, and other features indicative of a shallow carbonate shelf environment. It grades without any obvious stratigraphic break into the overlying Beekmantown Group.
Rocks of the Beekmantown Group are Ordovician in age and in the Lehigh Valley area fall into two mappable subdivisions, a lower unit consisting of thin to thick beds of gray fine to coarsely crystalline dolomite, the Rickenbach formation and an upper sequence of interbedded finely crystalline limestone and dolomite, the Epler formation. The combined thickness of these units exceeds 430 meters (1400 ft.) and the Beekmantown rocks are separated from the overlying Jacksonburg formation by an erosional unconformity.

The rocks making up the Jacksonburg formation are middle Ordovician in age and mark the transition from quiet carbonate shelf sedimentation to a deeper water turbidite environment. Jacksonburg rocks consist of limestone with up to 30 percent clay. They are for the most part, shaly rocks that show little tendency toward extensive solution and therefore mark the upper limit of rocks with which we are concerned in this report.

Markewicz and Dalton (1972) have made a study of cavern development in the carbonate rocks of New Jersey and have found that there is a stratigraphic control of the development of solution features in the rocks. In a sequence nearly identical to the stratigraphic column in the Lehigh Valley they have found that solution selectively follows coarser grain size and more lime-rich rocks so that units in the Leithsville, Beekmantown, and Allentown formations made up of coarse-grained dolomite or limestone are more prone to sinkhole development. No sinkholes of any consequence are found in the shaly Jacksonburg formation or in the fine-grained dolomites of the lower
units. They also found, as have many previous investigators, that fault zones are more prone to solution than areas of unfractured bedrock.

### 6.1.2 Weathering Characteristics and the Development of Sinkholes

The carbonate rocks of the Lehigh Valley have undergone a long history of weathering. In all probability, solution was initiated in carbonate rocks in this area at the close of the Mesozoic and has continued at varying rates since that time. The net result has been extensive solution of some of the more susceptible units in the carbonate rocks and the development of a thick residual soil over these units that commonly fills and tends to obscure the sinkholes.

Most of the sinkholes in the area have developed by solution along joint or fault planes and the bedrock soil interface is characterized by an irregular surface of pinnacles and depressions. Pinnacles represent areas of relatively low fracture density and consequently little solution where as depressions tend to develop over areas of closely spaced joints or other fractures.

The solution of carbonates leaves the insoluble constituents as residual soils which blanket the rock surface. Unless these insoluble constituents compose a large proportion of the rock, the fabric is destroyed and the soil is remolded. Subsequent erosion and reposition of this soil accounts for its great variability. The engineering properties of these soils may differ appreciably from those of sedimentary soils with the same composition and grain size.
distribution. Often, the properties vary laterally and with depth posing interesting problems for the geotechnical engineer.

Solution cavities are for the most part, filled with this residual soil, Fig. 5, and it is when this plug is partially removed that a sinkhole develops. This usually occurs when the hydrologic "equilibrium" of the area is upset in some manner. Excessive rainfall, a drop in the water table due to pumping or excavation, broken water mains, and the like, all can be correlated with sinkhole development. Foundering of the soil plug is the most common problem faced in any construction project, although differential compaction around a bedrock pinnacle can also pose serious problems. The only real solution lies in being aware of the presence of such features prior to construction. Many techniques have been devised to accomplish this, for example, probing, gravity studies, electrical resistivity studies, seismic studies and the like. These are beyond the scope of this paper but the reader is referred to the work of Bates (1973), who has published an excellent survey of the various techniques of detection as well as devising a very promising method of his own.

6.2 Foundation Construction Methods

It now becomes clear that the geologic conditions in a karstic environment generate problems for the geotechnical engineer. The ability to anticipate these problems and deal with them in the design stage of a project is of utmost importance if costs and construction schedules are to be kept within reason. Even if careful design
procedures are followed, unexpected problems may develop in the field due to rapid changes in the character of carbonate rock-soil interface. To illustrate some of the solutions to these problems, three case studies of foundations in the Lehigh Valley are presented. Each illustrate typical solutions utilized for the three major foundation systems: footings, piles, and caissons.

6.2.1 Deep Foundations

Conventionally, a deep foundation is defined as one in which the width of the foundation is less than its depth. Thus it is evident that caissons (drilled piers) and pile foundations fall into this category. Caissons have long been considered as the standard form of deep foundation to be used in problematic limestone areas. However, with proper precaution, H-piles may be used to transmit structural loads to a carbonate bedrock. The following sections will discuss examples of the use of each of these foundation techniques in the Lehigh Valley area. H-piles were used to support the multi-story municipal parking lot in Reading, Pennsylvania, while caissons provide the foundation for the Allentown-Sacred Heart Hospital in Salisbury Township.

CASE STUDY: CAISSONS, ALLENTOWN-SACRED HEART HOSPITAL

The Allentown-Sacred Heart Hospital was constructed in Salisbury Township of Lehigh County. The foundation consultant for this project was W. W. Lilly and the following discussion is based on his work (Lilly, 1973). The site is approximately 40 square hectares
(100 acres) and lies on the southwest quadrant of the intersection of state highways 309 and 29. The nine story structure pictured in Fig. 21 is of conventional steel and masonry construction but is underlain by the Allentown limestone. Thus, a detailed subsurface exploration had to be conducted in order to determine the type of foundation with which to support the structure. This consisted of a series of 36 borings which were made in order to determine the contour of the basement rock along with the properties of the overlying soil.

The investigation revealed that the basement rock pictured in Figs. 22 and 23 was part of the Allentown limestone. This Cambrian age formation is the largest of the limestone formations in the county, forming a continuous band from 1600 to 6400 meters (1 - 4 mi.) wide trending east-west across the county. Boring logs indicated that the site rested over the intensely weathered southern limb of an anticline with bedding planes dipping 10 to 12 degrees southward, see Fig. 24. A series of joint sets developed in the bedrock in response to stresses imposed on the bedrock during folding, see Fig. 25. These joints create drainage paths for groundwater flow and thus encourage the formation of sinkholes. Two existing surface depressions indicating possible sinkhole formation lie on the perimeter of the site and thus support this interpretation.

All of the surface features of a karst region are not observed in this general area for one major reason. Soil logs indicate that the Allentown limestone is covered by Illinoian glacial till. The
moraine remaining at the site following the retreat of the ice lobe has a width of 800 meters (0.5 mi.) and a relief of approximately 6 to 12 meters (20 to 40 ft.) above the outside district. The topography of the site (Lilly, 1973) "is of a subdued swell and sag with the swells suffering from considerable toning down as a result of relatively long exposure wash". Similar features are found elsewhere in the valley.

The soil at the site consists of an unstratified residual clay with glacially transported cobbles. There are also lenses of sands, rock fragments and boulders at the site resulting from the outwash of the retreating glacier. The boulders were found to be small enough for excavation without the use of explosives and groundwater observations taken at all borings indicated that no static water table existed. It was noted, however, that seepage water did accumulate in depressions on the basement limestone in various quantities.

With the results of the boring logs in hand, a shallow foundation was originally proposed. The foundation was to consist of spread and continuous footings with bearing pressures up to 200 kN/m² (400 psf). Based on the soil properties, this proposal would effectively resist any anticipated horizontal or uplift loads. Settlement also posed few problems. The problem of future karstic erosion after construction had not been dealt with, however.

Consequently, an alternate design utilizing caissons was proposed. This solution, though costly, would eliminate any differential settlement problems along with the major problem of further
sinkhole development. Since each caisson would be drilled down to firm bedrock which could be probed and inspected, this problem would in essence be eliminated. Triaxial tests were used to determine that the allowable bearing pressure of the Allentown limestone should be 1200 kN/m$^2$ (12 tsf) due to the extensively weathered character of the bedrock.

After a careful evaluation of each design, the caisson foundation was chosen. However, in an attempt to reduce cost, a study was made as to the actual cost evaluation involving risk. Results of this study indicated that an expanded full depth caisson founded on moderately weathered limestone would produce an economic solution with only a moderate risk of failure (Lilly, 1973). Figures 26 through 28 display the bottom conditions of a few of the caissons. Note the degree of weathering and jointing in each case.

During the construction period some problems involving groundwater were encountered. Near the bottoms of many of the caissons, groundwater had to be pumped out and into a nearby quarry. Since it was believed that the hydraulic gradients generated by pumping might effect the pouring of the caissons, it was decided that a tremie be utilized. Thus, where any inflow of water had occurred, the concrete was placed through the still water by use of this bottom dump bucket to a height sufficient to permit the balance of the concrete to be placed above water level by standard procedures. Problems still developed, however. The caissons that were poured into water were found by subsequent coring to be of inferior quality. The flow
of groundwater had segregated the concrete by washing out the fines from the mix. It was then decided to either grout or reinforce these defective caissons with steel rods depending on the results of cores taken at each pier. Grout filled in the void spaces left in the segregated concrete and steel rods provided extra structural strength when grout alone was found to be insufficient.

With these corrections completed, the foundation was deemed satisfactory and the hospital was completed. Experience from this project serves to display many of the "on the site" problems that exist with any foundation constructed in a limestone region. Figure 39 provides a table with which the design engineer may evaluate the advantages and disadvantages of a caisson foundation in a karst region. Comparison may also be made between pile foundations and spread footings.

CASE STUDY: H-PLIES, READING MUNICIPAL PARKING LOT

The Reading Municipal Parking Lot is located at Court and Seventh Streets in downtown Reading, Pennsylvania and is pictured in Fig. 29. It is a standard reinforced concrete three-deck structure founded on the Leithsville formation. This Leithsville bedrock prompted a thorough geotechnical investigation of the site.

Because of the previous experience in foundation construction in this area, borings were made at or near 37 of the proposed 48 column locations in order to explicitly determine subsurface conditions. After the borings were completed, the structural
configuration of the garage was altered so that the boring holes do not now coincide with the existing garage columns. Since such a large number of borings were made, however, no further seismic borings or resistivity surveys were deemed necessary. Figure 30 shows a typical boring log at the site while Fig. 31 portrays the bedrock contours.

It can be seen by viewing the cross-section in Fig. 32 that this is a pinnacled limestone region with the thickness of overlying soil varying drastically within the span of a few meters. Limestone boulders exist in the upper portions of the soil with soft silt below. Underlying the soil, layers of undependable dolomite, limestone and interbedded shales of the Leithsville formation can be found. Some voids exist in this bedrock and the bedrock-soil interface may slope up to 37 degrees. It is also noted that the groundwater table at this site is above the soil-bedrock interface.

Due to a combination of large structural loading, possibility of differential settlement, and danger of sinkhole formation, any form of shallow foundation was ruled out. Since the subsurface limestone was so irregular in hardness and in surface contour, it was decided that piles would be difficult if not impossible to drive. Therefore, a caisson foundation was first proposed for the structure.

The caisson foundation proposal proved to be too costly for the Reading Parking Authority. Since the groundwater table was relatively high with respect to the solution prone Leithsville formation, the risk of sinkhole formation was considered low. Therefore, an
investigation of the feasibility of a steel H-pile foundation was undertaken (Fisher - Fang Associates, 1974). Their study concluded that a pile foundation was feasible. However, pile tests were to be made in order to confirm their decision.

The increased use of H-piles in limestone regions is due to the ability of new driving systems which are capable of penetrating medium-hard rock without extensive driving damage to the pile. The use of thick sections with chamfered flanges and cast steel points reduce tip damage and also permit the pile to be driven straighter and deeper into the bearing strata. However, it was determined that there were several important factors concerning the use of piles at this site. Pile lengths could vary greatly (even within the width of a pile cap) due to erratic soil and rock conditions. Unreinforced pile tips could be damaged by boulders during driving. Piles driven into shale usually require redriving to seat the section. Finally, more lateral movement of piles and consequent variation of the piles from vertical should be expected relative to piles driven into a more uniform strata.

Thus, the objectives of the pile test program were to provide a basis for establishing:

1. Proper pile hammer size and type
2. Relationship between driven length and drill log information
3. Pile size
4. Point protection, if required
5. Allowable pile load
6. Allowance for lateral movement of pile top
7. Pile driving criteria

The test program conducted at two load setups located directly over some of the boring holes at opposite ends of the site. See Fig. 7 for test locations. Sixteen piles were driven including the central load test piles at each setup. Tests conducted including driving tests, load tests (both vertical and lateral), and extraction tests including inspection of piles after driving. Figure 30 displays the driving records and extracted piles at boring C11.

The results of the testing program confirmed that piles could successfully be used for the foundation with the following recommendations:

1. Piles require protective tips
2. Vertical piles should be utilized
3. Caps must allow for ± 152 mm (6 in.) tolerance in lateral movement of the pile cutoff point from the theoretical location
4. Termination of pile driving was to be 20 blows per inch (20 blows per 254 mm) by a 2490 - 2905 kg.-meter (18,000 - 21,000 ft.-lb.) hammer for the last 152 mm (6 in.).
5. All piles to be redriven a minimum of six hours after completion of initial driving to a blow count equal to that at the termination of
initial driving. If this may not be achieved in the first 152 mm (6 in.), continue until count is reached and redrive as specified above.

6. Minimum length of bottom section of piles is to be 914 cm (30 ft.)

7. All splices are to be 100% butt welds

These test results provide the necessary criteria for the geotechnical engineer to design a pile foundation in a karst region when he is faced with a high groundwater table and a fairly small amount of debris between the structure and the bearing layer of carbonate rock. The advantages and disadvantages of pile foundations in karst regions is summarized in Fig. 39, along with those of caissons and spread footings.

6.2.2 Shallow Foundations

In contrast to a deep foundation, a shallow foundation is defined as one in which the width of the foundation is greater than its depth. It is obvious that spread footings and mat foundations fall into this category. The low cost and ease of construction of these foundations make them attractive alternatives to piles or caissons. It is the relative inability of these foundations to resist karstic erosion that makes footings a questionable foundation form in limestone regions. With certain precautions, however, the risk of failure due to sinkhole development may be minimized. The
the following section will discuss the use of a shallow foundation on the St. Thomas More Catholic Church in Salisbury Township.

CASE STUDY: SHALLOW FOUNDATION FOR ST. THOMAS MORE CATHOLIC CHURCH

The St. Thomas More Catholic Church (Fig. 33) is located approximately one kilometer northeast of the Allentown Sacred Heart Hospital near the intersection of routes 309 and 29. Consequently, the geology of the two sites is basically the same. W. W. Lilly also became the foundation consultant at this site after the discovery of sinkhole problems. A spread footing foundation was proposed for this structure because of low cost and ease of construction. It was determined that sinkhole problems would be dealt with if any solutional activity was detected during excavation. Thus, no borings were taken in the underlying Allentown formation.

The only methods of dealing with sinkholes encountered during excavation come from engineering experience. Sowers (1975) outlined the basic techniques utilized in bridging sinkholes for spread footings. Each method is suited to different forms of sinkholes.

If solution pits are located and found to be shallow and cone-shaped, the hole is cleaned as deeply as possible and then filled with lean concrete forming a plug at least 1.5 times thicker than its width. A jackhammer is then used to probe the surrounding rock.
to determine its soundness. If the rock proves to be competent, the total capacity of the foundation will not be reduced significantly and construction may be continued with no design alterations.

Further problems develop if the solution pit is found to be wide and deep. The hole is first cleaned as deep as possible and plugged with lean concrete. Then, the foundation must be enlarged and reinforced to bridge the opening. If this enlargement creates a large eccentricity as in the case of many concentrated pits, the footings must often be joined to create a strap or mat foundation. This is necessary to prevent differential settlement.

No sinkholes were uncovered at the St. Thomas More Church foundation site but three developed under the adjoining pavement and sidewalk. The recent development of these sinkholes was believed by Lilly, 1970, to have been brought about by the alteration of runoff patterns during and after construction. Associated with the solution pits was a loss of bearing capacity and excessive settlement causing cracking of the pavements.

Excavation showed that the three sinkholes were about 4.25 m (14 ft.) deep and range in surface areas from 16 - 24 sq. meters (168 - 260 sq. ft.). The large sinkhole was cleaned out and filled with 12 cu. meters (16 cu. yds.) of concrete while the other two sinkholes, found to exist in a sandy-clay pocket within the limestone ledge, were filled with 34 metric tons of crushed stone. A subdrain was then recommended in order to prevent further sinkhole development.
from occurring. Figures 34 through 38 display the treatment of the large sinkhole from cleaning to plugging.

As can be seen by this short case study, there may be many hidden problems and related expenses in the use of spread footings in a karst region. The geotechnical engineer should insist on extensive subsurface investigations and rely on experience obtained from previous construction in the immediate area to minimize the problems that may be expected in a limestone terrain. Again, Fig. 39 presents the relative merits of spread footings relative to piles or caissons in a karst region.
I. **CALCAROUS MINERALS** - Predominant Mineral $\text{CaCO}_3$ (Calcite)

1. Limestone ($\text{CaCO}_3$): Specific Gravity 2.65 - 2.75
2. Dolomite ($\text{CaMg(CO}_3\text{_2})$): Half of the $\text{CaCO}_3$ replaced by $\text{MgCO}_3$. Specific Gravity 2.7 - 2.8
3. Dolomitic Limestones: Part of the $\text{CaCO}_3$ replaced by $\text{MgCO}_3$. Softer than sound igneous rock. Tends to become slippery if used for surface dressing due to physical and chemical polishing and to form rock powder if used in stone bases subject to heavy and frequent traffic loads. This powder makes such bases susceptible to frost. Becomes harder with increasing $\text{MgCO}_3$ content

II. **AVERAGE ENGINEERING PROPERTIES**

<table>
<thead>
<tr>
<th></th>
<th>Young's Modulus @ Zero Load $(x \times 10^5 \text{Kg/cm}^2)$</th>
<th>Bulk Density $(\text{g/cm}^3)$</th>
<th>Porosity (%)</th>
<th>Compressive Strength $(\text{Kg/cm}^2)$</th>
<th>Tensile Strength $(\text{Kg/cm}^2)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Limestone</td>
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<td>2.2-2.6</td>
<td>5-20</td>
<td>300-3500</td>
<td>50-250</td>
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<tr>
<td>Dolomite</td>
<td>4.0 - 8.4</td>
<td>2.5-2.6</td>
<td>1-5</td>
<td>800-2500</td>
<td>150-250</td>
</tr>
</tbody>
</table>

*Fig. 1 Characteristics and Engineering Properties of Carbonate Rock*

*Modified from Winterkorn and Fang, 1975*
Fig. 2 Schematic Diagram of a Collapse Sinkhole
(from Sowers, 1975)

Fig. 3 Collapse Sinkhole, Shelby County, Alabama
(from Garner, 1974)
Fig. 4 Schematic Diagram of a Funnel Sinkhole
(from Sowers, 1975)

Fig. 5 Cross Section of a Funnel Sinkhole
(courtesy of Dr. P. B. Myers)
Fig. 6 Subsidence Depression of Funnel Sinkhole

Fig. 7 Subsidence Depression of Funnel Sinkhole (courtesy of Dr. E. B. Evanson)
Fig. 8 Aerial Photo Showing Regional Jointing Pattern and Associated Sinkholes, Yugoslavia (courtesy of Dr. E. B. Evanson)
<table>
<thead>
<tr>
<th>GRAVITY</th>
<th>GROUND PENETRATING RADAR</th>
<th>EARTH RESISTIVITY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Detection capability is a function of anomaly size and density variation</td>
<td>Good for locating and measuring voids in some materials due to echoes generated from floor and roof of cavity</td>
<td>Good for locating both air and soil filled voids</td>
</tr>
<tr>
<td>Best for locating large voids or an area with many small voids</td>
<td>High search rate</td>
<td>Fair for void detection depending on characteristic seismic velocities</td>
</tr>
<tr>
<td>Penetration depth is good up to depths less than the diameter of an equivalent spherical cavity</td>
<td>Immediate data printout</td>
<td>Excellent for locating the groundwater table</td>
</tr>
<tr>
<td></td>
<td>Poor detection of 0.6 - 0.9 meter vertical pipe caverns</td>
<td>Vertical profiles may be obtained</td>
</tr>
<tr>
<td></td>
<td>Limited penetration depth in high loss residual soils</td>
<td></td>
</tr>
</tbody>
</table>

Fig. 9 Advantages and Disadvantages of Various Geophysical Exploration Techniques in Karst Regions

*Modified from Fountain (1976)
Fig. 10 Seismic Refraction Technique
(from Lowe, John III and Zacchjo, Philip, F., 1975)

Fig. 11 Reverse Profiling Technique
(from Dobrin, 1960)
Apparent Outline of Sinkhole

Fig. 12 Locating a Sinkhole by Fan Shooting from Two Shot Points A and B. Time Leads with Respect to "Normal" Curve are Plotted on Map (Top Portion) to Indicate the Location of the Sinkhole (from Dobrin, 1960)
Fig. 13 Resistivity Principles

Fig. 14 Schematic Diagram of a Resistivity Instrument
(from Soiltest, 1968)
Fig. 16 Plan View of Current Behavior in a Resistivity Survey (from Soiltest, 1968)
Fig. 17 Typical Profiling Survey Layout

The position of the center of the spread is fixed

Fig. 18 Typical Sounding Survey Layout (from Soiltest, 1968)
Fig. 19 Map of the Carbonate Rocks of the Lehigh Valley
(courtesy of Dr. P. B. Myers)
<table>
<thead>
<tr>
<th>Age</th>
<th>Rock Unit</th>
<th>Description</th>
<th>Thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lower &amp; Mid.</td>
<td>Shawangunk</td>
<td>Medium-gray to greenish-gray quartzites and quartz pebble conglomerates with</td>
<td>460 m (1500' ±)</td>
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<tr>
<td>Silurian</td>
<td>formation</td>
<td>minor shale beds (ridge former)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td><strong>UNCONFORMITY</strong></td>
<td></td>
</tr>
<tr>
<td>Mid. &amp; Upper</td>
<td>Martinsburg</td>
<td>Dark gray to medium dark gray claystone slate, gray-wacke siltstone and</td>
<td>3650 m (12,000'±)</td>
</tr>
<tr>
<td>Ordovician</td>
<td>formation</td>
<td>carbonaceous shale</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td><strong>UNCONFORMITY</strong></td>
<td></td>
</tr>
</tbody>
</table>
| Middle            | Jacksonburg            | Dark gray to black high-calcium and argillaceous limestone (little evidence  | 150-250 m (500'  |}
| Ordovician        | limestone              | of solution)                                                                 | to 800')        |
|                   |                        | **UNCONFORMITY**                                                            |                 |
| Lower             | Beekmantown            | Light to dark medium gray dolomite and interbedded limestone and dolomite    | 440 m (1435')   |
| Ordovician        | Group*                |                                                                               |                 |
|                   |                        | **UNCONFORMITY**                                                            |                 |
| Upper             | Allentown              | Light to dark medium gray stromatolitic dolomite and oolitic dolomite        | 520 m (1700')   |
| Cambrian          | dolomite*              |                                                                               |                 |
|                   |                        | **UNCONFORMITY**                                                            |                 |
| Middle            | Leithsville            | Light to dark gray dolomite interbedded with light-gray to tan phyllite      | 300 m (1000')   |
| Cambrian          | formation*             |                                                                               |                 |
|                   |                        | **UNCONFORMITY**                                                            |                 |
| Lower             | Hardyston              | Tan to medium gray ortho-quartzite, arkosic sandstone and quartz-pebble      | 30 m (up to     |
| Cambrian          | quartzite              | conglomerate with minor shale                                               | 100')           |
|                   |                        |                                                                               |                 |
| Precambrian       | Gneiss                 | Complex of granitic, and amphibolitic gneisses with minor metasediments and  |                 |
|                   |                        | metavolcanies                                                                |                 |

Fig. 20 Rocks of the Lehigh Valley

Modified from Drake (1965) and Drake and Epstein (1967).
*Rock units subject to extensive solution
Fig. 21 The Allentown Sacred Heart Hospital Center

Fig. 22 Folded and Weathered Allentown Dolomite
Fig. 23 Massively Bedded and Jointed Allentown Dolomite

Fig. 24 Cross Section of Subsurface Conditions at the Allentown Sacred Heart Hospital Center
(modified from W. W. Lilly)
ISOMETRIC VIEW OF AN ANTICLINE showing the development of characteristic joint sets. (1) Conjugate Joints, (2) Cross Joints or Extension Fractures, (3) Longitudinal Joints, and (4) Radial Tension Joints. These fractures channel the flow of groundwater and thus promote the development of sinkholes in carbonate rock.

Fig. 25 Joint Sets in an Anticline
Fig. 26 Unweathered Caisson Bottom at Allentown Sacred Heart Hospital Center
(from Fisher-Fang Associates, 1974)

Fig. 27 Weathered and Jointed Caisson Bottom at Allentown Sacred Heart Hospital Center
(from Fisher-Fang Associates, 1974)

-68-
Fig. 28 Weathered and Jointed Caisson Bottom at Allentown Sacred Heart Hospital Center (from Fisher-Fang Associates, 1974)

Fig. 29 The Reading Municipal Parking Lot
### Driving Records: Test at Boring C11 (Blows per foot)

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<th>DEPTH</th>
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Fig. 30 Typical Boring Log and Driving Records at Reading Municipal Parking Lot (original data from Fisher-Fang Associates, 1974, then from APF Bulletin HPP 752)
Fig. 31. Bedrock Contours at Reading Municipal Parking Lot (original data from Fisher-Fang Associates, 1974, then from APF Bulletin HPP 752).
Fig. 32 Cross Section of Subsurface Conditions at the Reading Municipal Parking Lot (original data from Fisher-Fang Associates, 1974, then from APF Bulletin HPP 752)

Limestone boulders in upper strata caused unprotected pile ends to spread so they could not be driven through underlying silts to good bearing.
Fig. 33 The St. Thomas More Roman Catholic Church

Fig. 34 Cleaning of Sinkhole at St. Thomas More Roman Catholic Church
(courtesy of W. W. Lilly)
Fig. 37 Concrete Plug Being Placed in Sinkhole at St. Thomas More Church
(courtesy of W. W. Lilly)

Fig. 38 Concrete Plug Being Placed in Sinkhole at St. Thomas More Church
(courtesy of W. W. Lilly)
<table>
<thead>
<tr>
<th>CRITERION</th>
<th>FOOTINGS</th>
<th>CAISSONS</th>
<th>PILES</th>
</tr>
</thead>
<tbody>
<tr>
<td>1) Availability of construction contractors</td>
<td>Many</td>
<td>Few</td>
<td>Fair Amount</td>
</tr>
<tr>
<td>2) Equipment required for installation</td>
<td>Minimal</td>
<td>Large Amount</td>
<td>Small Amount</td>
</tr>
<tr>
<td>3) Consolidation of lower layers</td>
<td>None</td>
<td>Some</td>
<td>Quite'a Bit</td>
</tr>
<tr>
<td>4) Type of load transfer on bearing surface</td>
<td>End Bearing</td>
<td>Mostly End Bearing</td>
<td>End and Friction Bearing</td>
</tr>
<tr>
<td>5) Penetration through debris</td>
<td>Good</td>
<td>Good</td>
<td>Fair</td>
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<tr>
<td>6) Depth Restrictions</td>
<td>5 Meters</td>
<td>50 Meters</td>
<td>100 Meters</td>
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<tr>
<td>7) Resistance to horizontal loads</td>
<td>Good</td>
<td>Good</td>
<td>Very good with battered piles</td>
</tr>
<tr>
<td>8) Problems with groundwater</td>
<td>Dewatering Required</td>
<td>Dewatering or drill casing</td>
<td>None</td>
</tr>
<tr>
<td>9) Problems with solutinal erosion</td>
<td>Bearing surface may be</td>
<td>Footing bearing surface may</td>
<td>Can not tell</td>
</tr>
<tr>
<td></td>
<td>inspected. Sinkholes</td>
<td>be inspected.</td>
<td>exactly what bearing</td>
</tr>
<tr>
<td></td>
<td>require special treatment</td>
<td></td>
<td>strata is</td>
</tr>
<tr>
<td></td>
<td>and design alterations</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10) Cost for medium depth foundation (&lt;5 meters)</td>
<td>Low</td>
<td>High</td>
<td>Medium</td>
</tr>
<tr>
<td>11) Cost for deep foundation (&lt;7 meters)</td>
<td>High</td>
<td>Highest</td>
<td>Lowest</td>
</tr>
</tbody>
</table>

Fig. 39 Advantages and Disadvantages of Various Foundation Systems in Karst Regions
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