Effect Of Horizontal Piles On The Soil Bearing Capacity For Circular Footing Above Cavity

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EFFECT OF HORIZONTAL PILES ON THE SOIL BEARING CAPACITY FOR CIRCULAR FOOTING ABOVE CAVITY

by

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B.S. Peruvian University of Applied Sciences, 2001

A thesis submitted in partial fulfillment of the requirements for the degree of Master of Science in the Department of Civil and Environmental Engineering in the College of Engineering and Computer Science at the University of Central Florida Orlando, Florida

Spring Term
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ABSTRACT

The design of foundation in normal soil conditions is governed by bearing capacity, minimum depth of foundation and settlement. However, foundation design in karst regions needs to consider an additional criterion associated to the possibility of subsurface subsidence and ravelling sinkholes. Under this environment, alternative techniques are needed to improve the subsurface soil.

In this study general background information is given to understand the geological characteristics of Central Florida and why this area is considered to be a karst region and susceptible to sinkholes formation. Traditional foundation design techniques on karst regions are addressed in this paper. Finally, the use of a network of three subsurface horizontal piles is proposed and the effect on stress increase and soil bearing capacity for footing due to the horizontal piles is investigated. Finite element computer software is used to analyze the stress distribution under different conditions and the results are discussed.

The objective of this study is to determine whether or not horizontal piles under a circular footing at the sinkhole site is a viable solution to reduce the stress increase in the soil induced by the footing load. The horizontal piles located at a certain depth below the center of the footing intercepts the cone of pressure due to the footing load. Also, it is the purpose of this research to determine the effect on the soil bearing capacity for footing due to the proposed horizontal piles at the sinkhole prone area.
In 1983 Baus, R.L and Wang, M.C published a research paper on soil bearing capacity for strip footing above voids. In their research, a chart for soil bearing capacity for strip footing located above a void was presented. However, in this paper we present a chart for circular footing size as a function void location and a design chart for circular footing size with a network of three underground piles.

The result indicates that with the horizontal piles placed above the cavity, the stress increase caused by the footing load substantially decreases as compared to the situation of no horizontal piles, thus increases the soil bearing capacity for the normal design of footing size.

The approach of using the horizontal piles placed in between the footing and the subsurface cavity is a new concept that has not been experienced previously. The results are strictly based on the analytical model of finite element program. Before full implementation for the construction practice, further research and experimental work should be conducted.
The writer of this thesis wishes to express his sincere appreciation and gratitude to his major professor, Dr. Shiou-San Kuo, for his guidance, assistance and infinite patience during the preparation of his thesis. The writer would also like to express special thanks to Dr. Lei Zhao for his important support, valuable time, and being constantly available for addressing coursework issues and counseling during the course of his graduate studies. The writer would also like to express special thanks to Dr. Manoj Chopra for his valuable time to be thesis committee to review my paper.

Finally, he would like to thank his family, his friends from the structures lab and especially his wife for the support during his two years of graduate studies.
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<tr>
<td>FT</td>
<td>Feet</td>
</tr>
<tr>
<td>LB</td>
<td>Pounds</td>
</tr>
<tr>
<td>PSF</td>
<td>Pound per square foot</td>
</tr>
<tr>
<td>KSI</td>
<td>Kilo pounds per square inch</td>
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<tr>
<td>γ</td>
<td>Specific weight of the soil in psf</td>
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<td>μ</td>
<td>Poisson ratio</td>
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CHAPTER ONE: INTRODUCTION

Florida’s landscape is characterized by the presence of many lakes of different sizes. Many of these lakes were formed by sinkholes over thousands of years, such as Lake Eola in downtown Orlando. Florida is therefore a region subjected to this type of sinkhole activity characteristic of karst regions, especially Central Florida. This karst topography is in the areas where the limestone lays in the underground as subsurface bedding. The chemical erosion of the underground limestone, over thousands of years, results in cavities which eventually collapse or become filled by ravelling of overlying soils.

There are many good effects which can result from sinkholes. For example, sinkholes create lakes than can be used for recreational activities. Lakes also provide food and enhances the landscape, also some sinkholes has the capability to recharge the aquifer. While there are the positive effects, there are also negative effects resulting from sinkhole activities, such as structure damages and property losses.

The extents of the damage caused by a sinkhole can range from a simple hole in the backyard, to the complete destruction of a house, damage to utilities, roadways and dam failure. The cost of the damage may be in the order of hundreds of thousands of dollars, or even worse, as it can cause injuries or loss of lives. An example of the danger behind sinkholes can be seen in the case of two employees of the City of
Portland when a large sinkhole swallowed the truck they were driving, causing both men to be severely injured.

A lot of research and efforts have been made to find methods to successfully detect underground cavities. One of the methods to detect underground cavities is a geophysical technique known as ground penetrating radar (GPR). Ground penetrating radar is a non-destructive method in geotechnical investigation capable to detect underground conditions which may lead to the development of a sinkhole. Therefore, the detection of the subsurface cavity is not a major issue. The major problem is the construction and design considerations when building structures in sinkhole prone regions. Different alternatives are available for foundation design when building structures in the karst regions. These may include grouting the cavity with concrete, or driving the deep foundation piles through the cavity. The positive and negative aspects of these methods will be briefly discussed in Chapter Two.

The use of horizontal piles under a footing are proposed as an alternative in this research study. The horizontal piles will be considered as a network of three piles in an axisymmetric model. The objective of this research is to determine whether or not the horizontal piles under a circular footing can reduce the stress increase, thus increasing the soil bearing capacity for footing over the subsurface cavity, commonly filled with very loose muddy soil.
Because of the complexity of modeling a subsurface cavity and the placement of horizontal piles over the cavity, the finite element method of analysis was performed to investigate the stress increase and soil bearing capacity behavior of a circular footing situated above a soft muck or void and subjected to static vertical central loading.

The extraordinary technology currently available has resulted in advanced software products for engineering and scientific analysis. The availability of these products makes it possible to use powerful techniques, such as a finite element analysis. However, finite element software does not inherently lead to good results; it depends on the guidance of the user to obtain useful and meaningful results. In J.P Carter’s (Krahn, 2004) keynote address at the GeoEng2000 Conference in Melbourne, Australia, he spoke about rules for modeling and stated that modeling should start with an estimate. The methodology used is thoroughly explained in Chapter Three and it is mainly based in cross checking the results with closed form geotechnical equations to validate the finite element software results and then adding the more complex factors such as the subsurface cavity and horizontal piles.

As stated in the above paragraph, a finite element axisymmetric model was created using Sigma/W (a geotechnical finite element software). The model consisted of a cross section of a cohesionless soil profile subjected to the action of a vertical load from a circular footing. The depth of foundation was five feet below the surface. Eight cases were analyzed to determine the effect of the horizontal piles in the distribution of the vertical stress increase due to the footing load. The objective of this research is to study
the effect of horizontal piles placed beneath the foundation with a subsurface cavity existing.

The first case, Case 1, consisted only of the soil profile without any external loading. An in-situ analysis was used to establish the initial overburden stresses induced by the self weight of the soil. The second case, Case 2, considered the addition of the footing load and a load / deformation analysis was performed to find the resulting stress changes when the footing load was applied. Until this point we had not yet considered the void. The third case, Case 3, included a void of ten feet in diameter under the footing at a depth of twenty feet below the bottom of the footing. The fourth case, Case 4, increased the size of the void to fifteen feet and the depth below the foundation decreased to fifteen feet. The fifth case, Case 5, increased the size of the void to twenty feet in diameter and decreased the depth below the foundation to only ten feet.

The finite element model was run for each case and results were obtained regarding the effect of the void in the stress increase on the soil mass compared to the no void condition. Then, Case 3 was modified to include the horizontal piles considered as a network of three piles in an axisymmetric model, placed at a depth of three feet below the bottom of the foundation, as this would be Case 6. The same was done to Cases 4 and 5, for which horizontal piles were established, redefining both models to be Cases 7 and 8. The finite element model was run again for each of the cases and the results were compared with each other to determine the effect of the horizontal piles on the vertical stress of the soil and in the soil bearing capacity for footing.
CHAPTER TWO: LITERATURE REVIEW

Karst regions and Geological aspect of Central Florida

Slovenia is the home of Kras region, sometimes referred also as Classical Karst. Kras is a limestone plateau located on the eastern side of the Adriatic Sea, delimited by the valleys of the rivers Soca to the northwest, Vipava to the northeast, Pivka to the east, and Reka to the southeast. It is about 25 miles long and 8 miles wide. A broader area, between Ljubljana (Slovenia) and Trieste (Italy) is generally referred to as Classical Karst.

At Karst in Slovenia, the limestone rocks are honeycombed by tunnels and openings dissolved out by ground water and by the underground drainage. Irregular topography of this kind, developed by solution of surface and ground waters, is known as karst topography. The special landforms of karst include sinkholes, dry valleys, cave systems and springs (Waltham, 2005). This region became the focus of first scientific research of karst morphology and hydrology. That is why Kras (Carso in Italian, or Karst in German) was the area which gave the scientific name to all karsts in the world (Skinner, 1972).

In the United States, karst regions exists in central Tennessee, southern Indiana, Alabama, Kentucky, Florida, Texas, Ohio, Pennsylvania and New Mexico.
Sinkholes are a characteristic feature of karst processes in Central Florida and they are very common due to the geological formation. Central Florida subsurface includes the Ocala limestone, the Avon Park limestone and the Hawthorn formations. The uppermost layer consists of unconsolidated deposits of sand and clay in varying thickness, from a few inches to 40 feet. Below this layer is the Hawthorn formation, which consists of clay, sandy clay, sand, dolomite and limestone. The Hawthorn formation is a semi-permeable to highly impermeable layer. This layer is relatively thin and in some Florida areas it is non existent (Skinner, 1972).

The Ocala limestone formation is located below the Hawthorn formation. It is the oldest formation penetrated by most wells. Well records indicate that the top of the Ocala formation is from 100 to 150 feet beneath the surface. The Ocala formation is pure limestone and parts of it are porous and contain solution channels that permit free circulation of groundwater (Ardaman, 1969). The Ocala formation is rated as having a “moderately high transmissibility”.

The Avon Park limestone formation is generally below the Ocala formation and is underlined by basement rock. The Avon Park formation is rated as, “overall transmissibility very high”. 
Limestone is a carbonate rock composed largely of the mineral calcite (Stokes, 1978). Natural water contains carbonic acid, which reacts with calcite to form calcium bicarbonate, a soluble substance that is carried away in solution. Calcium bicarbonate is approximately 30 times more soluble in water than calcium carbonate; for that reason, the carbonation reaction causes increasingly rapid dissolution of the limestone (Stokes, 1978).

Groundwater that flows through pores and along fractures lines chemically erodes the limestone rock by solution. Large voids in the limestone are formed as a result of concentrated groundwater flowing through areas where the underground limestone is more porous or has a higher solubility. The solution process continues enlarging these cavities (Sweeney, 1986).

Sinkholes are typically formed by roof collapse of a cavity in the limestone, or by raveling collapse due to cavity development in unconsolidated overburden. Sinkholes are commonly circular depressions of different depth and diameters. Most common sinkholes have diameters that range from 5 to 20 feet, however sinkholes of over 100 feet in diameter have been recorded. Figure 1 shows a colossal sinkhole that developed in Winter Park, Florida, in May, 1981. The picture shows the effect of a large limestone cavity and a thick, unconsolidated overburden. Figure 2 shows a home destroyed by a sinkhole in the Bartow area of Polk County, in the late 1960’s and Figure 3 is another example of damaged caused by sinkholes.
Figure 1: Sinkhole in Winter Park, Fl 1981 (Beck, 1986)

Figure 2: House destroyed by sinkhole in Polk County (Beck, 1986)
The most common sinkholes occurring in Florida are subsidence or ravelling sinks. These occur in regions where unconsolidated overburden covers the dissolved cavities in the limestone. As the void grows to a point near the ground surface, overlying deposits collapse resulting in a sinkhole. Ravelling failures are probably the most dangerous of all subsidence phenomena associated with limestone because they develop suddenly and without notice (Sowers, 1975).

Different conditions may cause the sinkhole collapse, but the most common cause is due to the increase of filtration through the clay layer. It is well known that sinkhole collapse can be induced by variations in the relationship between the water table and
the potentiometric surface. This variation may be caused by heavy pumping, intense pumping, or increased localized surface infiltration, such as below sewage ponds. In Florida, a high percentage of sinkholes occur during dry seasons when well draw–down leads to an increase in the groundwater infiltration through the clay layer. The raveling process is enhanced by this heavy infiltration.

Another effect that lowering the water table has in the formation of sinkholes is based on the concept of the addition of a “new” load to the ground surface. This “new” load is expressed in units of force per area. This load is actually the increase in effective pressure which is caused by eliminating the buoyancy effect of the water. In other words, the unit weight for a sandy soil, not necessarily submerged, could be assumed to be 125 pcf, the unit weight of water is 62.5 pcf and under buoyant conditions the effective weight of the soil would be the difference between 125 pcf and 62.5 pcf or 62.5 pcf. Therefore, as the water table is lowered by each foot, it will cause an increase of approximately 62.5 pcf in effective unit weight. If the water table is lowered ten feet this will result in an effective pressure increase of $10 \times 62.5 = 625$ psf. A sinkhole is then formed simply because the roof over the million year old cavity is not capable of supporting the additional 625 psf.
Design and construction of foundations in limestone terrain may face the inherent defects and weakness of the soil and rock. The first step in designing a building foundation in central Florida is to locate the building away from the influence of potential sinkholes.

In order to evaluate sites for sinkhole potential it is necessary to evaluate historical data on sinkhole activity and obtain information about the geology as well as hydrogeology characteristic of the area. Geophysical methods, particularly ground-penetrating radar, supplemented with conventional exploration techniques, are the primary tools used in locating areas of potential sinkhole activity within a specific site (Sweeney, 1986).

In general, the investigation of sinkhole activity starts with studying the geology and hydrogeology of the region, also mapping historical sinkholes that have occurred in the project vicinity. If the new structure can be located to avoid all of the suspect areas and is outside the zone of influence of a probable sinkhole, then foundation exploration and design goes on normally. If the case of a building has to be located close to or over a suspected sinkhole, then a more thorough underground exploration program is required and a special design of foundation must be addressed.
Precise measurement of the water table is of extreme importance on sites that are being investigated for sinkhole activity. Depressions in the water table are indicators of breaches in the confining layer. To detect this water table behavior, a shallow monitor well is commonly left at each test hole location for determining the direction of groundwater flow on the site (Sowers, 1996).

If after a thorough investigation of a site does not reveal any sinkholes or raveled conditions above the confining layer, then the building foundation design proceeds normally. On the other hand, if these conditions are discovered with sinkhole potential and the building has to be at the sinkhole site, it is recommended to either design the foundation to span a potential sinkhole or plug the breach. In most cases grouting the breach is not an economic alternative and should only be considered in very unusual circumstances (Waltham, 2005).

Design of foundations for major structures in the sinkhole prone areas presents unique challenges. The proposed tasks may include the following:

a) Rock improvement by grouting: This is a viable option when the rock mass is riddled with small cavities and when the rock cavities are relatively free of soil filling. By filling a large proportion of the cavities with Portland cement grout it is possible to improve the strength and reduce its compressibility. Other benefits of grouting, it reduces the possibility of collapsing of small caves, and also reduces the erosion of overburden soil into cavities and the development of further erosion of the overburden soil. However, it is very common that not all the primary porosity will be filled. In terms of cost, rock
improvement by grouting is a very expensive technique mainly because of the cost of introducing the material into the cavities through several drill holes. Also, it is a very expensive technique because a significant amount of the Portland cement grout is wasted by its flow through the larger cavities into areas of no concern at the site and sometimes off the site into properties owned by others. Groundwater flow is also affected as the normal water flow through the soil and cavities is forced to seek new paths (Waltham, 2005).

b) Deep foundation on rock: Deep foundation directly supported on competent rock is considered when the overburden soil is thick, but not strong enough for supporting the expected loads with a level of safety and within tolerable settlement. Also, this type of foundation design is considered where the risk of soil ravelling dome collapse is great, or when the upper surface of the rock cannot sustain the foundation loading. It appears that deep foundations may be the best solution when dealing with sinkhole areas; however there are some problems and risks regarding the use of deep foundation. For instance, it is difficult and sometimes impossible to inspect the critical areas of these foundations. These critical areas include the area of the foundation in contact with the rock, the rock adjacent to and immediately below the foundation, and the condition of the deeper rock in the zone of significant stress increase from the foundation loads (Sowers, 1996).

c) Pile foundation: Most of piles that are currently used for foundation design have been used in limestone terrain. The capacity of the piles is a combination of the side shear or
skin friction along the pile shaft with the adjacent soil above the rock and the end bearing capacity on the rock below the pile tip. Some of the disadvantages or risks when using pile foundation are that piles that are driven onto a sloping rock surface often slide downward; either bending or breaking and sometimes they curl up or buckle (Sowers, 1996). In some other cases the pile may break the bridging above the cavity and triggering the collapse of the ground surface.

d) Drilled shafts: Drilled shafts, also known as caissons, are used for deep foundation solutions to high axial and / or lateral loads action. Drill shafts are built by digging out the soil inside a large tubular steel shaft, then inserting a huge “cage” made up of steel reinforcing rods, and pumping in concrete from the bottom up. As the pour proceeds, the steel tubing is slowly removed. The side friction of the soils against the surface of the concrete shaft should provide most of the capacity. The other capacity comes from the end bearing of the shaft on the soil below it.

e) Geogrid: Synthetic plastic reinforcement is commonly used in roadway construction. It is a more economical solution compared to placing concrete slabs where the soil and the sub base undermining by sinkholes is a probable threat. Geogrid can be rolled out during sub base construction to cover any area prone to sinkhole failure (Waltham, 2005).

f) Horizontal piles: Horizontal piles proposed in this research study are drilled holes in horizontal level at certain depth below the footing and then filled it with concrete. The
stress increase will be dissipated as it encounters the horizontal piles that are placed above cavities or soft layers of soil. This option is investigated and it is discussed in the following section of this paper.

**Horizontal piles**

Many techniques are available for excavating horizontal holes without disrupting the surface. These methods include a) Horizontal boring, b) Small diameter directional drilling, c) Large diameter directional drilling, d) Microtunnelling, e) Pipe jacking, and f) Conventional tunneling.

We will focus on the first one, which is horizontal boring. Horizontal boring includes excavating miniature tunnels by mechanical tooling. The methods are classified as non man-entry systems. There are two techniques in horizontal boring: Auger method and Slurry method. The vertical alignment can be changed during the boring process, but the horizontal alignment is not adjustable and depends on the initial alignment, as well as the ground conditions.

The horizontal auger earth boring method is a process of simultaneously jacking and casing through the earth while removing the spoil inside the encasement by means of a rotating auger. The auger is driven by a power source in the entrance pit which transmits power to the cutting head. As the auger proceeds, pieces are added until it
exits into the exiting pit. Steerable cutting heads on the end of the boring casing along with a grade-sensing device allows for extreme accuracy in holding grade tolerances.

The casing also serves to support the soil around it as the spoil is removed. Typical diameters range from 4 to 84 inches, with driving lengths up to 600 feet. Figure 4 shows the sketch of horizontal auger boring. Figure 5 and Figure 6 show the equipment of typical auger boring and the emerged cutter head assembly, respectively.

Figure 4: Sketch of auger boring setup (Najafi, M., and Gunnink, B. 2005)
Figure 5: Typical auger boring equipment (Najafi, M., and Gunnink, B. 2005)

Figure 6: Emerged cutter head assembly (Najafi, M., and Gunnink, B. 2005)
The Slurry method uses drill bits and drill tubing instead of cutting heads and augers. A slurry mixture is used to keep the drill bit clean and assist in the spoil removal. The fluid is not used to cut the face of the tunnel; cutting is done mechanically. Figure 7 shows the equipment setup for slurry method.

![Slurry System Diagram]

Figure 7: Typical Slurry system (Najafi, M., Gunnink, B. and Davis, G. 2005)

After the horizontal hole is excavated, concrete is then pumped inside the hole making it a horizontal concrete pile. The depth of horizontal piles should be calculated based on: 1) footing size and stress distribution, 2) the soil conditions, and 3) the location of subsurface cavity, before the horizontal hole is augered.

The horizontal concrete piles will have an effect in reducing the stress increase, thus increasing the soil bearing capacity for footing. Figure 8 depicts the distribution of stress.
in the soil profile without the horizontal pile and Figure 9 shows the horizontal pile existed within the stress zone induced by footing load. The horizontal pile located at a certain depth below the center of the footing intercepts the cone of pressure; this is shown in Figure 10.

\[ Q = \text{Column Load} \]

Figure 8: Stress distribution in soil profile under the footing load
Figure 9: The placement of horizontal pile and stress redistribution under the pile.

Figure 10: Horizontal pile placed within stress zone induced by footing load.
CHAPTER THREE: METHODOLOGY AND MODELING

Because of the complexity of the analysis when dealing with an underground void and with the addition to the model of horizontal piles, the finite element method (FEM) was used. FEM was performed to investigate the stress increase within soil mass and soil bearing capacity behavior for a circular footing situated above a subsurface cavity or soft muck within a cavity.

In application of FEM; the circular footing and the soil profile were created for two dimensional axisymmetric model using the finite element software. A network of three underground horizontal piles was assumed to represent for the axisymmetry of the problem. The first step was to create a simple finite element model of the soil profile that did not include a footing was generated and its stresses calculated by the finite element software. These stresses were the in-situ stresses of the soil (stresses due to the overburden load). These model stresses were then compared with the results from theoretical closed form geotechnical equations. After verification by the closed form solutions, the validity of the model was determined, and more complicated parameters were incorporated.

Once the footing was included, and before the addition of the void and horizontal piles to the model, it was possible to determine the stress increase and the soil bearing capacity with the finite element software and again verify the results by theoretical geotechnical equations. Once more a conclusion was drawn to the relative proximity
between the model results generated by the FE software and the results from the closed form solution. The reliability of the model was again determined.

Once the void was added to the model, we compared the soil bearing capacity results to soil bearing capacity calculated from geotechnical equations and therefore verified if the results from the FE software were in agreement to the expected values.

Modeling and theoretical comparisons have been the main research methodologies employed when the concrete piles were not present. However, because of the complexity of the model that includes the concrete piles, only model evaluations were used in the post horizontal piles introduction.

Modeling the soil, footing, and horizontal piles configuration in a computer helped us understand the effect on the distribution of the stress increase in the soil, and conclusions were obtained regarding the effect that the addition of the horizontal piles had in the stress increase. Also, it helped us determine the effect on soil bearing capacity for footing in the presence of an underground void and the effect on bearing capacity in the horizontal piles above a subsurface cavity situation.

The approach of using the horizontal piles placed in between the footing and the subsurface cavity is a new concept that has not been experienced previously. Because of the model complexity and this being the first research study on this topic, it is beyond the scope of this investigation to derive an equation that can relate the stress increase
in a footing, soil, void, and concrete pile system. The results are strictly based on the analytical model of finite element program. Before full implementation for the construction practice, further research and experimental work should be conducted.

In this chapter we explained the different cases that were modeled and analyzed, and then in Chapter Four we will show the results. As discussed previously, the objective is to determine the behavior of the stress increase and soil bearing capacity in the presence of an underground cavity before and after the addition of horizontal piles.

In this study, the horizontal piles were placed at a depth of 3 ft below the foundation and were considered as a network of three piles for an axisymmetric model as shown in Figure 11. The location of the horizontal piles may depend on the site conditions, such as soil properties, depth, and size of subsurface cavity. The depth of foundation, $D_f$, as well as the properties of the soil remained constant. The dimensions of the void and the depth of the void from the bottom of the footing varied. Figure 12 shows the geometry and characteristic of the study case.
Figure 11: Network of three piles in an axisymmetric model

Network of three piles in axisymmetric model

Figure 12: Model case study

Dense Uniform Sand
\(\gamma_{\text{dry}} = 105 \text{ lb/ft}^3\)
\(E_s = 8000 \text{ lb/in}^2\)
\(\mu_s = 0.30\)
\(e = 0.45\)

8 ft (diameter)

Q = 500,000 lb

5 ft

B_d = 3 ft

50 ft

100 ft

Cavity
\(\gamma = 5 \text{ lb/ft}^3\)
\(E_s = 100 \text{ lb/in}^2\)

Network of three horizontal piles
The soil was modeled as a linear elastic material for the purpose of studying the overburden stresses and the stress increase due to the footing load. When studying the effect of the cavity in the soil bearing capacity for the footing, an elastic-plastic model was preferred. In the elastic-plastic model stresses are directly proportional to strains until the yield point is reached. The importance of using this type of model was that the FE software uses the Mohr-Coulomb yield criterion which allowed us to identify when the soil had reached its failure condition, thus the soil bearing capacity for footing could be computed.

The first step to verify the validity of the software was to compare the in-situ overburden stress from both finite element model and hand calculation with closed form equations. Subsequently, seven more cases were generated from the model as shown in Table 1 for the scopes of this study.

Note that the size of the cavity reflects the depth of the cavity. For Cases 3 and 6, the depth of the cavity below the footing is 20 feet for 10 foot diameter of void (see Figure 15). For Cases 4 and 7, the depth of the cavity is 15 feet while in Cases 5 and 8, the depth of the cavity becomes 10 feet. The properties of soil that may commonly be representative to Central Florida are given in Table 2.
Table 1: Case Scenarios

<table>
<thead>
<tr>
<th>Case</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>In-situ stresses only</td>
</tr>
<tr>
<td>2</td>
<td>Footing load</td>
</tr>
<tr>
<td>3</td>
<td>In-situ stresses + Footing load + Void (10 ft diameter)</td>
</tr>
<tr>
<td>4</td>
<td>In-situ stresses + Footing load + Void (15 ft diameter)</td>
</tr>
<tr>
<td>5</td>
<td>In-situ stresses + Footing load + Void (20 ft diameter)</td>
</tr>
<tr>
<td>6</td>
<td>In-situ stresses + Footing load + Void (10 ft diameter) + Horizontal pile</td>
</tr>
<tr>
<td>7</td>
<td>In-situ stresses + Footing load + Void (15 ft diameter) + Horizontal pile</td>
</tr>
<tr>
<td>8</td>
<td>In-situ stresses + Footing load + Void (20 ft diameter) + Horizontal pile</td>
</tr>
</tbody>
</table>

Table 2: Soil properties

<table>
<thead>
<tr>
<th>Description</th>
<th>Symbol</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit weight of soil (pcf)</td>
<td>$\gamma$</td>
<td>115</td>
</tr>
<tr>
<td>Angle of internal friction (degrees)</td>
<td>$\phi$</td>
<td>34</td>
</tr>
<tr>
<td>Modulus of Elasticity (psi)</td>
<td>$E$</td>
<td>8,000</td>
</tr>
<tr>
<td>Poissons Ratio</td>
<td>$\mu$</td>
<td>0.300</td>
</tr>
<tr>
<td>Initial Pressure Coefficient</td>
<td>$k_o$</td>
<td>0.429</td>
</tr>
</tbody>
</table>

For the subsurface cavity, it was assumed that a very soft muck with very low unit weight is within the void. The unit weight was considered as 5 pcf, the friction angle of 0 and modulus of elasticity 100 psi. Figure 13 shows the model created with the finite element software in Case 1 to calculate the in-situ stresses and compare the results with hand calculation closed form equations. The comparison values for the two methods will be described in the next chapter.
Figure 13: Case 1 of Finite element model for calculating the in situ stresses

Once the load was applied on the ground, the stress increased within the soil mass. The application of a load on the surface of a soil mass causes increases in stress within the soil. The magnitude of stress increase due to load application decreases with increasing depth and at some depth becomes insignificant. The increase in stress due to the footing load was analyzed in Case 2 of this research.

In Case 2, the footing load was applied to the model, and the results of model stress increase were also compared with results by hand calculations. Meanwhile in this second case, we obtained the bearing capacity at normal conditions without subsurface
void. A column load of 500,000 pounds applied to the footing was used for this study. The footing pressure of the 8 feet circular footing can be calculated as the following.

\[ Q = 500,000 \text{lb} \]

\[ A = \frac{\pi \times D^2}{4} = \frac{\pi \times (8)^2}{4} = 50.27 \text{ft}^2 \]  \hspace{1cm} (1)

\[ q = \frac{Q}{A} \]

\[ q = \frac{500,000}{50.27} = 9947.18 \text{psf} \]  \hspace{1cm} (2)

Where

\( A \) = area of a circular footing with diameter 8 ft.

\( q \) = footing pressure

Figure 14 shows the finite element model to determine the stress increase due to the footing load.
In Case 3, a ten feet diameter void was created in the finite element software. Also in this case, we requested the bearing capacity from the finite element output and compared the results with a closed form solution given by Meyerhof and Hanna (Das, 2002) for the layered soils with a strong layer over a weak layer. The rationale for this comparison was that the compacted soil mass beneath the footing was assumed as a strong layer, while the muck soil filled in the cavity was assumed as a weak layer. Figure 15 presents the finite element model for Case 3.
In Case 4, the void diameter was increased to fifteen feet, while the location of the void was closer to the bottom of the footing. In addition, we requested the bearing capacity of the soil from the FE output and again compared the results with a closed form solution given by Meyerhof and Hanna (Das, 2002) for layered soils with a strong layer over a weak layer. Figure 16 presents the finite element model for Case 4.
In Case 5, the void diameter was further increased to twenty feet, and the location of void was even closer to the bottom of the footing. Also in this step, the bearing capacity of soil was requested from the finite element output and used for comparison from the solution given by Meyerhof and Hanna (Das, 2002). Figure 17 presents the finite element model for Case 5. The purpose of varying the size and depth of the cavity was to understand the effect of the soil bearing capacity due to the depth and size of the cavity.

Figure 16: Case 4 of Finite element model from footing load plus 15 feet void at 15 feet depth.
Cases 6 through 8 were repeated from Cases 3 through 6 with addition of horizontal piles placed at a depth of 3 feet below the footing. The purpose for placing the horizontal piles from Cases 3 to 6 was to understand the effect of horizontal piles on the reduction of stress increase and increase of soil bearing capacity due to the existing of cavity. The finite element models for Cases 6 through 8 are shown in Figure 18 through 20 respectively. The results will be described in the next chapter.
Figure 18: Case 6 of Finite element model with 10 feet void at 20 feet depth and horizontal pile
Figure 19: Case 7 of Finite element model with 15 feet void at 15 feet depth and horizontal pile
Figure 20: Case 8 of Finite element model with 20 feet void at 10 feet depth and horizontal pile
CHAPTER FOUR: FINDINGS

Results of in-situ stresses and increase in stress on Cases 1 and 2

The in-situ stresses (overburden pressures) were determined in Case 1. The results from the finite element software were verified with hand calculations. The results presented in Table 3 were based on the properties of uniform soil given in Table 2 (pp. 38). The computed results of the vertical total stress at a point are equal to the unit weight of soil times the depth, as can be seen from the following equation:

$$\sigma_v = \gamma \times d$$  \hspace{1cm} (3)

Table 3 displays the model's results and the hand calculations. Both methods gave the same results.

Table 3: Comparison of results of Case 1 from model and hand calculations

<table>
<thead>
<tr>
<th>Depth (ft) from surface</th>
<th>In situ stresses (psf)</th>
<th>Model</th>
<th>Equation 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>1</td>
<td>115.00</td>
<td>115.00</td>
<td>115.00</td>
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<td>2</td>
<td>230.00</td>
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<td>3</td>
<td>345.00</td>
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<td>4</td>
<td>460.00</td>
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<tr>
<td>10</td>
<td>1,150.00</td>
<td>1,150.00</td>
<td>1,150.00</td>
</tr>
<tr>
<td>15</td>
<td>1,725.00</td>
<td>1,725.00</td>
<td>1,725.00</td>
</tr>
<tr>
<td>20</td>
<td>2,300.00</td>
<td>2,300.00</td>
<td>2,300.00</td>
</tr>
<tr>
<td>25</td>
<td>2,875.00</td>
<td>2,875.00</td>
<td>2,875.00</td>
</tr>
<tr>
<td>30</td>
<td>3,450.00</td>
<td>3,450.00</td>
<td>3,450.00</td>
</tr>
<tr>
<td>35</td>
<td>4,025.00</td>
<td>4,025.00</td>
<td>4,025.00</td>
</tr>
</tbody>
</table>
Figure 21, further depicts the stress contours plotted from the model output. Figure 22 shows a graph of in-situ stresses from Table 3.

Figure 21: Case 1 of stress contour of in-situ stresses
Figure 22: Comparison of results of in-situ stress obtained with FEM and hand calculation

Case 2, describes the stress increase, $\Delta \sigma$, within the soil mass due to the footing load. The results from the FEM were verified by the hand calculations as shown in Table 4. The stress increase, $\Delta \sigma_z$, at any point below a uniformly loaded circular area was calculated by the following equation:

$$\Delta \sigma_z = q \times (A + B) \tag{4}$$

Where, A and B are functions of $z/R$ and $r/R$ and can be found in any geotechnical text book. The terms $z$, $r$ and $R$, are defined as shown in Figure 23.
Figure 23: Vertical stress at any point below a uniformly loaded circular area (Das, 2004)

For the stress increase at the center of the uniformly loaded circular area, $r = 0$. The results from Case 2 obtained with the finite element model and Equation 4 is presented on Table 4 for comparison.
Table 4: Comparison of results of Case 2 (Stress Increase) from model and hand calculation

<table>
<thead>
<tr>
<th>Depth (ft) from bottom of footing</th>
<th>Stress increase (psf)</th>
<th>Offset from center of footing (ft)</th>
<th>0</th>
<th>B/2</th>
<th>B</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model</td>
<td>Equation 4</td>
<td>Model</td>
<td>Equation 4</td>
<td>Model</td>
<td>Equation 4</td>
</tr>
<tr>
<td>0</td>
<td>9,969.50</td>
<td>9,947.18</td>
<td>4,973.63</td>
<td>4,973.59</td>
<td>45.78</td>
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<td>9,583.90</td>
<td>9,805.27</td>
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<td>4,571.43</td>
<td>55.87</td>
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<td>8,787.50</td>
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<td>3,962.87</td>
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<td>7,561.10</td>
<td>7,798.59</td>
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<td>3,645.74</td>
<td>397.66</td>
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<td>6,194.10</td>
<td>6,430.32</td>
<td>3,116.38</td>
<td>3,304.75</td>
<td>523.21</td>
</tr>
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<td>20</td>
<td>523.77</td>
<td>568.32</td>
<td>485.93</td>
<td>519.14</td>
<td>402.84</td>
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<td>25</td>
<td>388.88</td>
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<td>369.84</td>
<td>352.33</td>
<td>325.85</td>
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<tr>
<td>30</td>
<td>312.93</td>
<td>259.48</td>
<td>246.13</td>
<td>251.66</td>
<td>240.49</td>
</tr>
</tbody>
</table>

Table 4 shows that the results by both methods agree with each other within an acceptable tolerance.

Figure 24, shows the plot of curves from the data in Table 4. The figure also clearly reveals that the outputs from the model agreed with the hand calculated results using Equation 4. From the results either in Table 3 and 4 or Figures 22 and 24 the confidence in using FEM was confirmed.
Figure 24: Comparison of stress increases obtained from the finite element model and hand calculation

The stress increase contours from the model output within the soil mass due to the 500,000 lb column load is shown in Figure 25. The stress increase contours from other column loads can also be generated from $\Delta \sigma/q$ versus depth. Stress increase at any point within the soil mass can be determined from the stress contours.
Figure 25: Case 2 of stress contours of stress increase due to footing load

Results of stress increase on Cases 3, 4 and 5

In Cases 3 through 5, a ten feet, fifteen feet, and twenty feet diameter void, respectively, was created in the model. The void was located beneath the footing at a depth of twenty feet for Case 3, at fifteen feet for Case 4 and ten feet for Case 5 as shown in Figure 26 through 28. The results of stress increase distribution from the finite element output for Cases 3 through 5 are presented on Tables 5 through 7, respectively. In these tables a comparison between stress increase with and without subsurface cavity was presented.
From the results shown on the tables the stress increase within the soil with void is higher than without void. However, as we get closer to the top of the void the stress increase decreases because of the difference in modulus of elasticity between the soil layers. As expected at the location of the cavity, due to the lower modulus of elasticity of the muck soil filled in the void, the stress increase decreases significantly. Note that in Tables 5 through 7 the location of the void was indicated with a shaded pattern. However, for Cases 3 and 4 were the void size is 10 feet and 15 feet, respectively, the results of stress increase at an offset of B from center of footing was not indicated with shaded pattern simply because the void is not contained in that area.

Figure 26: Case 3 FEM from footing load and 10 foot void at depth of 20 feet.
Figure 27: Case 4 FEM from footing load plus 15 foot void at depth of 15 feet.
Figure 28: Case 5 of FEM from footing load plus 20 foot void at depth of 10 feet.
Table 5: Results of Case 3 (Stress Increase) through FEM

<table>
<thead>
<tr>
<th>Depth (ft) from bottom of footing</th>
<th>Offset from center of footing (ft)</th>
<th>With Void</th>
<th>Without Void</th>
<th>With Void</th>
<th>Without Void</th>
<th>With Void</th>
<th>Without Void</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
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</tbody>
</table>

Stress increase (psf)

Table 6: Results of Case 4 (Stress Increase) through FEM

<table>
<thead>
<tr>
<th>Depth (ft) from bottom of footing</th>
<th>Offset from center of footing (ft)</th>
<th>With Void</th>
<th>Without Void</th>
<th>With Void</th>
<th>Without Void</th>
<th>With Void</th>
<th>Without Void</th>
</tr>
</thead>
<tbody>
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<td>1</td>
<td></td>
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<td></td>
</tr>
<tr>
<td></td>
<td>2</td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td></td>
<td>3</td>
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</table>

Stress increase (psf)
Table 7: Results of Case 5 (Stress Increase) through FEM

<table>
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<tr>
<th>Depth (ft) from bottom of footing</th>
<th>Stress increase (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>9,986.50</td>
</tr>
<tr>
<td>1</td>
<td>9,657.01</td>
</tr>
<tr>
<td>2</td>
<td>8,865.65</td>
</tr>
<tr>
<td>3</td>
<td>7,645.65</td>
</tr>
<tr>
<td>4</td>
<td>6,290.73</td>
</tr>
<tr>
<td>5</td>
<td>4,750.35</td>
</tr>
<tr>
<td>10</td>
<td>52.87</td>
</tr>
<tr>
<td>15</td>
<td>25.37</td>
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<tr>
<td>20</td>
<td>13.90</td>
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<td>25</td>
<td>7.47</td>
</tr>
<tr>
<td>30</td>
<td>5.40</td>
</tr>
</tbody>
</table>

Results of bearing capacity on Cases 2, 3, 4 and 5

The ultimate soil bearing capacity formula for a circular footing was derived by Terzaghi in 1943 and is given by the following equation:

\[ q_u = 1.3 \cdot c \cdot N_c + q \cdot N_q + 0.3 \cdot \gamma \cdot B \cdot N_\gamma \]  

(5)

Where

\( q_u = \) ultimate bearing capacity,
\( c' = \) cohesion,
\( q = \gamma \times D_r \)
and $N_c$, $N_q$, and $N_γ$ are bearing capacity factors contributed to cohesion, surcharge and unit weight, respectively. The values of these factors are given in the tables from any geotechnical engineering textbook. For $\phi = 34$ the values of $N_c$, $N_q$, and $N_γ$ are 52.64, 36.50 and 38.04 respectively. Equation 5 can be computed to find the bearing capacity, $q_u$, for Case 2:

\[
q_u = 1.3 \cdot 0 \cdot 52.64 + 115 \cdot 5 \cdot 36.50 + 0.3 \cdot 115 \cdot 8 \cdot 38.04
\]

\[
q_u = 31,486.54 \text{psf}
\]

In the finite element model, the ultimate bearing capacity is computed based on Equation 7:

\[
q_u = \frac{F \cdot 2 \cdot \pi}{\pi \cdot D^2} \cdot \frac{4}{4}
\]

(7)

Where,

$F = \text{Ultimate force from the Y- Boundary force versus displacement diagram obtained from the model output shown on Figure 29 for Case 2, Figures 31 through 33 for Cases 3 through 5 and Figures 35 through 37 for Cases 6 through 8.}$

$D = \text{Diameter of the footing,}$
With $F$ obtained from Figure 29 for Case 2, the ultimate bearing capacity of the footing from Equation 7 was computed as shown below:
\[ q_u = \frac{260,000 \cdot 2 \cdot \pi}{\pi \cdot 8^2} = 32,500 \text{ psf} \]

The ultimate bearing capacity computed by Equation 5 from Terzaghi shows a slightly lower number than by Equation 7 from the model. However, the difference is insignificant. The confidence of using the model to compute the bearing capacity value was also evident.

From Cases 3 through 5 with the condition of a subsurface cavity, the bearing capacity was requested from the finite element output and compared with the hand calculations. In this situation, the ultimate bearing capacity of a circular footing with a void underneath can be interpreted as a layered soil condition with a strong layer over a weak layer given by Meyerhof and Hanna in 1978 (Das, 2002).

Under this assumption, if the top strong layer is relatively thick, the failure surface in soil under the foundation will be located inside the stronger layer. For this case the ultimate bearing capacity is given by Terzaghi:

\[ q_u = q_{u(t)} = \gamma_1 \cdot D_f \cdot N_{q(1)} + 0.3 \cdot \gamma_1 \cdot B \cdot N_{q(t)} \] (8)

Where

\[ \gamma_1 = \text{unit weight of top layer} \]

\[ N_{q(1)} \text{ and } N_{q(t)} = \text{bearing capacity factor, function of internal friction angle, } \phi \text{ of the top layer of soil (Das, 2002)} \]
On the other hand, if the thickness of the stronger layer under the foundation is relatively thin, the failure in the soil would take place by punching in the stronger layer followed by a general shear failure in the bottom weaker layer or muck soil (Das, 2002). For this condition, the ultimate bearing capacity is given as:

\[
q_u = q_{u(b)} + 2 \cdot \gamma_1 \cdot H^2 \cdot \left(1 + \frac{2 \cdot D_f}{H}\right) \cdot \left(\frac{K_s \cdot \tan\phi_1}{B}\right) \cdot \lambda_s - \lambda_{s_1} \cdot H \leq q_{u(t)} \tag{9}
\]

Where

\(K_s\) = punching shear coefficient

\(\lambda_s\) = shape factor

\(q_{u(t)}\) = ultimate bearing capacity calculated by Equation (8)

\(q_{u(b)}\) = ultimate bearing capacity of the bottom soil layer using Equation (10).

\[
q_{u(b)} = \gamma_1 \cdot (D_f + H) \cdot N_{q(2)} + 0.3 \cdot \gamma_2 \cdot B \cdot N_{\gamma(2)} \tag{10}
\]

Where

\(\gamma_2\) = unit weight of the lower layer or weak layer soil

\(N_{\gamma(2)}\) = bearing capacity factor of the lower layer or weak layer soil
The value for the shape factor can be taken to be approximately 1 (Das, 2002). The punching shear coefficient, $K_s$, can be obtained from Figure 30. Equation 9 is used for hand calculation to compare with the results by FEM.

Figure 30: Variation of $K_s$ with $(\gamma_2 N_{\gamma(2)}) / (\gamma_1 N_{\gamma(1)})$ (Das, 2002)
As for the results of bearing capacity by FEM, outputs for Cases 3 through 5 were presented on Figures 31 through 34, respectively. These figures show the force versus displacement under the footing until the yield criterion is reached, thus giving the ultimate force. Then, by solving Equation 7 on page 60 the bearing capacity of the model is calculated.

Figure 31: Case 3 Y-Boundary Force versus Displacement for Bearing Capacity
(Sigma/W, 2004)
Figure 32: Case 4 Y-Boundary Force versus Displacement for Bearing Capacity
(Sigma/W, 2004)
The FEM results on bearing capacity and hand calculations were presented on Table 8. To further illustrate the agreement between model and hand calculations, from the data on Table 8 a bar chart was generated and shown on Figure 34. As seen from Table 8
and Figure 34, both methods yield similar results and, once again, give us a high level of confidence in the use of the finite element software in this research study.

Table 8: Result of bearing capacity on Cases 3 through 5

<table>
<thead>
<tr>
<th>Case 3 (model)</th>
<th>Case 3 (hand calculation)</th>
<th>Case 4 (model)</th>
<th>Case 4 (hand calculation)</th>
<th>Case 5 (model)</th>
<th>Case 5 (hand calculation)</th>
</tr>
</thead>
<tbody>
<tr>
<td>16,873.00</td>
<td>15,119.00</td>
<td>10,998.00</td>
<td>9,665.00</td>
<td>6,249.00</td>
<td>5,423.00</td>
</tr>
</tbody>
</table>

Figure 34: Comparison of bearing capacity results by finite element model and Equation 9
The effect of bearing capacity due to the existing of horizontal pile may be considered a fairly complex case for geotechnical engineering. No research study has been done in the past. Therefore, use of FEM analysis was needed for this case.
Results of stress increase and bearing capacity on Cases 6, 7 and 8

In Cases 6 through 8, the stress increases due to the footing load were obtained from the model outputs. The results were compared with Cases 3 through 5 and presented in Tables 9 through 11. The dash lines shown in Tables 9 through 11 for the Cases 6 through 8 represent the locations of the horizontal piles. The location of the void is indicated with a shaded pattern. However, for Cases 6 and 7 were the void size is 10 feet and 15 feet respectively, the results of stress increase at an offset of B from center of footing is not indicated with the shaded pattern simply because the void is not contained in that area.

Interestingly enough, the tables reveal that the stress increase above the horizontal pile have been only slightly affected by the placement of the pile. These results can only be found through the FE model. Nevertheless, this is not part of scope of the investigation.

However, the stress increase is dissipated as it encounters the network of horizontal piles. The stress increase below the location of the piles is reduced by 50% to 60% as compared with the non-horizontal piles situation.
Table 9: Comparison of results of Case 6 with Case 3 from model outputs

| Depth (ft) from bottom of footing | Offset from center of footing (ft) | Stress increase (psf) | | | | |
|---------------------------------|-----------------------------------|----------------------|---|---|---|---|---|---|
|                                 | 0                                  | B/2                  | B | | | |
|                                 | With pile (Case 6) | Without pile (Case 3) | With pile (Case 6) | Without pile (Case 3) | With pile (Case 6) | Without pile (Case 3) |
| 0                               | 9,944.70                         | 9,991.59             | 5,170.56                 | 5,037.52             | 56.87               | 55.89               |
| 1                               | 9,989.56                         | 9,705.24             | 5,281.65                 | 4,500.79             | 67.29               | 65.78               |
| 2                               | 9,330.56                         | 8,884.41             | 4,325.10                 | 4,022.10             | 370.56              | 346.70              |
| 3                               | 8,286.40                         | 7,602.59             | 3,898.56                 | 3,617.14             | 566.26              | 412.58              |
| 4                               | 3,012.70                         | 6,230.60             | 1,451.00                 | 3,215.32             | 358.36              | 543.25              |
| 5                               | 1,924.90                         | 4,830.56             | 1,220.36                 | 2,888.98             | 350.98              | 630.58              |
| 10                              | 841.92                           | 1,720.26             | 639.80                   | 1,520.72             | 289.44              | 826.48              |
| 15                              | 286.01                           | 660.65               | 260.56                   | 766.21               | 223.41              | 581.55              |
| 20                              | 6.64                             | 13.11                | 5.45                     | 11.39                | 199.63              | 425.10              |
| 25                              | 3.59                             | 7.38                 | 2.51                     | 5.48                 | 153.95              | 290.69              |
| 30                              | 1.92                             | 4.50                 | 1.37                     | 3.70                 | 110.42              | 230.47              |

Note: = Location of horizontal pile

Table 10: Comparison of results of Case 7 with Case 4 from model outputs

| Depth (ft) from bottom of footing | Offset from center of footing (ft) | Stress increase (psf) | | | | |
|---------------------------------|-----------------------------------|----------------------|---|---|---|---|---|---|
|                                 | 0                                  | B/2                  | B | | | |
|                                 | With pile (Case 7) | Without pile (Case 4) | With pile (Case 7) | Without pile (Case 4) | With pile (Case 7) | Without pile (Case 4) |
| 0                               | 9,984.90                         | 9,982.50             | 5,489.60                 | 5,030.15             | 64.90               | 64.58               |
| 1                               | 9,847.56                         | 9,662.10             | 4,699.23                 | 4,502.63             | 85.65               | 79.65               |
| 2                               | 9,646.00                         | 8,855.60             | 4,335.26                 | 4,048.45             | 310.56              | 299.45              |
| 3                               | 8,307.30                         | 7,638.50             | 3,875.65                 | 3,614.63             | 663.56              | 408.69              |
| 4                               | 2,880.20                         | 6,233.25             | 1,406.90                 | 3,199.13             | 300.56              | 533.68              |
| 5                               | 1,788.60                         | 4,890.33             | 1,309.00                 | 2,866.12             | 250.43              | 605.89              |
| 10                              | 502.48                           | 1,585.62             | 490.15                   | 1,460.14             | 229.23              | 801.44              |
| 15                              | 12.04                            | 25.40                | 10.76                    | 24.91                | 120.34              | 250.56              |
| 20                              | 6.77                             | 13.65                | 5.62                     | 12.07                | 105.56              | 230.78              |
| 25                              | 3.62                             | 7.44                 | 2.50                     | 6.12                 | 92.34               | 175.45              |
| 30                              | 1.95                             | 4.99                 | 1.61                     | 4.33                 | 70.23               | 130.48              |

Note: = Location of horizontal pile
Table 11: Comparison of results of Case 8 with Case 5 from model outputs

<table>
<thead>
<tr>
<th>Stress increase (psf)</th>
<th>Offset from center of footing (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0</td>
</tr>
<tr>
<td>Depth (ft)</td>
<td>With pile (Case 8)</td>
</tr>
<tr>
<td>from bottom of footing</td>
<td>B/2</td>
</tr>
<tr>
<td>0</td>
<td>9,988.63</td>
</tr>
<tr>
<td>1</td>
<td>9,865.36</td>
</tr>
<tr>
<td>2</td>
<td>9,584.20</td>
</tr>
<tr>
<td>3</td>
<td>8,223.10</td>
</tr>
<tr>
<td>4</td>
<td>2,302.80</td>
</tr>
<tr>
<td>5</td>
<td>1,624.00</td>
</tr>
<tr>
<td>10</td>
<td>22.82</td>
</tr>
<tr>
<td>15</td>
<td>11.70</td>
</tr>
<tr>
<td>20</td>
<td>5.36</td>
</tr>
<tr>
<td>25</td>
<td>2.74</td>
</tr>
<tr>
<td>30</td>
<td>1.74</td>
</tr>
</tbody>
</table>

Note: = Location of horizontal pile

The results on bearing capacity for the Cases 6 through 8 from the finite element model were also compared with the bearing capacity for the Cases 3 through 5. The ultimate bearing capacity from the finite element model was computed based on the F factors generated by Figures 35 through 37. Again, these figures show the force versus displacement under the footing, giving the ultimate force.
Figure 35: Case 6 Y-Boundary Force versus Displacement for Bearing Capacity

(Sigma/W, 2004)
Figure 36: Case 7 Y-Boundary Force versus Displacement for Bearing Capacity
(Sigma/W, 2004)
Figure 37: Case 8 Y-Boundary Force versus Displacement for Bearing Capacity

(Sigma/W, 2004)

Then, using Equation 7 on page 59 and Figures 35 through 37, the bearing capacity was calculated. Table 15 presents the results from Cases 6 through 8 compared against those from Cases 3 through 5.
Table 12: Comparison of results on soil bearing capacity of Cases 6 through 8 with Cases 3 through 5

<table>
<thead>
<tr>
<th>Case 3 (without piles)</th>
<th>Case 6 (with piles)</th>
<th>Case 4 (without piles)</th>
<th>Case 7 (with piles)</th>
<th>Case 5 (without piles)</th>
<th>Case 8 (with piles)</th>
</tr>
</thead>
<tbody>
<tr>
<td>16,873.00</td>
<td>24,400.00</td>
<td>10,998.00</td>
<td>17,248.00</td>
<td>6,249.00</td>
<td>9,999.00</td>
</tr>
</tbody>
</table>

The results on Table 12 showed that the presence of the network of three horizontal piles affects the ultimate soil bearing capacity for footing. When the void is located at 20 feet depth (Case 6 and Case 3) the horizontal piles increased the bearing capacity by 45%. When the void was closer to the footing as in Cases 7 and 8, the increase in bearing capacity was more significant, 57% and 60%, respectively. The reason is that a shallow subsurface cavity reduces significantly more the soil bearing capacity for footing as compared to the normal condition where no cavity existed.

The results are somewhat surprising. This significant increase in soil bearing capacity is the prospective objective of this research study. Example analysis illustrated in later section proves that the initial designed size of footing may not be increased significantly to still be in safe side over the cavity with the placement of a network of three horizontal piles as proposed in this study. However, the FEM computer software is only a theoretical model. An experimental work may be conducted in future research to verify the theoretical result.
Summary of Case results by plotting charts

A series of charts are presented to better visualize the results. These charts were prepared with the results of the finite element model and show the positive effect that the addition of the horizontal piles had on the decrease of the stress increase in the soil mass. Figures 38, 39, and 40 shows the stress increase below the center of the footing under the different cases. It can be seen from the figures that the stress increase below the piles was reduced to approximately fifty to sixty percent. But, the existing of horizontal piles causes a slightly increase of stress increase between the bottom of the footing and the horizontal pile.

![Graph showing stress increase](image)

Figure 38: Summary of stress increase for Cases 1, 3 and 6 (10 foot diameter void at 20 feet below bottom of footing)
Figure 39: Summary of stress increase for Cases 1, 4 and 7 (15 foot diameter void at 15 feet below bottom of footing)
Figure 40: Summary of stress increase Cases 1, 5 and 8 (20 foot diameter void at 10 feet below bottom of footing)

Similarly, a chart was also plotted to show the effect on the bearing capacity due to the void under the eight foot circular footing for both cases of without horizontal piles condition and with the horizontal piles. Figure 41 shows the comparison of bearing capacities with and without the network of three horizontal piles. As stated previously for the Table 12, the presence of horizontal piles had increased the ultimate bearing capacity from 45% to 60% for Case 6 to Case 8.
Figure 41: Comparison of soil bearing capacities with and without single horizontal pile under eight foot circular footing

Since the soil bearing capacity depends greatly on size and location of the void, and the placement of horizontal piles, therefore, we can generate the relationships between soil bearing capacity for footing and footing size, B, depth to void, D, and void size, W, using finite element model. The void size, W, and the depth to void, D, were combined with the footing width into a dimensionless ratio. The soil bearing capacities of the footings with voids were expressed as a percentage of the bearing capacity of without void condition. Figure 42 presents the results of the plots.
Figure 42: Soil bearing capacity for footing as a function of void size and location

Figure 44 shows that for a given W/B, the effect of a void on bearing capacity decreases as D/B increases. For example, a 24 foot diameter size of subsurface cavity located at the depth 40 feet under the footing of 8 foot diameter. The soil bearing capacity under the normal (no void) condition is calculated per say as 5,000 psf. From Figure 41, D/B = 40/8 = 5, W/B = 24/3 = 3, \( q_{u(v)/q_u} = 0.68 \), one can find the soil bearing capacity with given cavity will be reduced to be 0.68 x 5,000 = 3,400 psf. If now the void is at 60 feet under the footing from Figure 41, D/B = 60/8 = 7.5, W/B = 24/3 = 3, \( q_{u(v)/q_u} = 0.85 \), thus the soil bearing capacity will be 0.85 x 5,000 = 4,250 psf.
The trend of these graphs were in agreement with Figure 43 from the study done by Baus, and Wang, on bearing capacity of strip footing above voids (Baus and Wang, 1983). In their research, they concluded that the bearing capacity becomes smaller when the void is located closer to the footing and that the effect of the void on bearing capacity decreases as the depth to void increases.

![Diagram of bearing capacity](image)

**Figure 43:** Bearing capacity of strip footing as a function of void size and location (Baus and Wang, 1983)

Furthermore, from the finite element output, design charts for circular footing above void without horizontal piles and with underground horizontal piles were presented on Figures 44 and 45, respectively. From these very useful design charts, the diameter of a
circular footing placed above a subsurface cavity without horizontal piles and with proposed network of horizontal piles can be easily computed. In the development of these charts a safety factor of 1 was considered. Examples of footing design with and without horizontal piles using the design charts are illustrated in the following section. Important to mention that the factor D from figures refers to the location of the void and as stated previously, on Chapter 3, the size of cavity reflects the depth of cavity.

Figure 44: Design chart for circular footing size as a function of void location without horizontal pile
Ultimate load $Q$ as a function of void location (with network of three piles)

Figure 45: Design chart for circular footing size as a function of void location with network of three horizontal piles

With the addition of the horizontal piles, an increase in the soil bearing capacity was evident. It is very common to find that soil bearing capacity is regarded as a fixed value for the particular soil data, such as particle size, density, and shear strength parameters.

Soil density may be unintentionally altered either favorably or unfavorably; also it may be deliberately changed by a planned preconstruction program. Bearing capacity may also be affected by the injection of chemical admixtures or other additives in the soil.
The effect of the horizontal piles on the soil bearing capacity for footing was principally due to the decrease of the stress induced from the footing load and as a second factor, the concrete horizontal piles provides a greater stiffness to the soil mass under the footing.

The examples of two conditions are given to illustrate the application of the charts and the effect that underground voids has in footing dimension and allowable column load to a footing. For both conditions the column load would be 600,000 lbs and as previously stated for the use of the charts, the Factor of Safety must be 1 for using charts from Figures 44 and 45.

**Example for Condition 1 (without horizontal piles):**

Determine the footing size required to support the column load of 600,000 lbs when a subsurface cavity is existed at depth of 10 feet below the footing.

From Figure 44, with column load of 600,000 lbs and curve D = 10, the footing diameter would be 14 feet.

**Example for Condition 2 (with network of horizontal piles):**

Determine the footing size required to support the column load of 600,000 lbs when a subsurface cavity is existed at depth of 10 feet below the footing and network of piles are placed at 3 feet below the footing bottom.
From Figure 45, with column load of 600,000 lbs and curve D = 10, the footing diameter would be 9.0 feet.

From the results of these two examples it is clearly shown that the presence of the subsurface cavity greatly affects the footing size and that by the addition of the horizontal piles the dimension of the footing drastically decreases to almost normal condition values.
CHAPTER FIVE: SUMMARY AND RECOMMENDATIONS

Central Florida due to its geological underground formation is considered as a region subjected to sinkhole activity which can result in structure damages and property losses, drastically affecting the lives of people. A lot of research has been done to find methods to successfully detect underground cavities; however, techniques to minimize sinkhole collapse still are relatively new areas in research study.

In this research study, a new technique for footing designs in the sinkhole prone areas was presented. The objective of the research study was to determine whether or not the proposed technique of underground horizontal piles under a circular footing can reduce the stress increase, thus increasing the soil bearing capacity for footing over the subsurface cavity, commonly filled with very loose muddy soil.

Literature review has shown different alternatives that are available for foundation design when building structures in the karst regions. The positive and negative aspects of these methods were discussed in Chapter Two. Also, In Chapter Two, the proposed method of horizontal piles under a footing was presented as an alternative for footing design in sinkholes areas. Two techniques of horizontal boring were briefly discussed; the Auger method and the Slurry method. However, there are available a wide variety of methods for the installation of horizontal piles.
In Chapter Three the methodology and modeling used in this research study was presented. The complexity of the analysis when dealing with an underground void and with the addition to the model of horizontal piles required us to use the finite element method (FEM). An axisymmetric model was created with the finite element software and different conditions for footing placed above an underground cavity were considered. Modeling and theoretical comparisons were the main research methodologies employed and the results between model and closed form solution showed good agreement.

From Chapter Four, which discussed the outputs of the FEM for the in-situ stresses and stress increase of the non void condition, we can conclude that the results agree well with the closed form solution. Similarly, from the outputs of soil bearing capacity by the finite element model before considering the void and after considering the void, the results also agree well with the closed form solution. The use of finite element model to analyze the stress increase and bearing capacity behavior has proven to be a satisfactory approach.

The results from the cases with the shallow underground network of three horizontal piles installed above subsurface cavity show the effective dissipation of the stress increase, and increase of soil bearing capacity for footing.

The design charts as shown in Figures 44 and 45 present the relationship between the location of subsurface void reflected by size, allowable column load and footing size on the factor of safety of 1, with and without the installation of horizontal piles. The
purposes of Figures 44 and 45 are so that in the case a subsurface sinkhole cavity is discovered, they can be used to investigate the soil bearing capacity due to the effect of the cavity, and the installation of number and depth of horizontal piles can be a quick remedial solution before total structure failure.

The alternative of using the horizontal piles placed in between the footing and the subsurface cavity as a means to dissipate the stress increase is a new concept that has not been tested. The results are strictly based on the analytical model of finite element program and therefore it is recommended that further research and experimental work should be conducted before full implementation in construction practice.
APPENDIX A: A TYPICAL MODEL INPUT AND OUTPUT DATA
Figure 46: Material Properties typical input interface

Figure 47: Soil Unit Weight typical input interface
Figure 48: Typical Model interface

Figure 49: Output interface
Figure 50: Analysis Settings interface
Figure 51: Typical Force versus Displacement Output
LIST OF REFERENCES


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Sigma/W for stress and deformation analysis (Ver. 5 for Windows) [Computer software]. Alberta, Canada: Geo-Slope International Ltd., 2004.


