geopier®

GEOPIER®

Foundation
and
Soil Reinforcement
MANUAL

Authors:
Nathaniel S. Fox, P.E., Ph.D.
Michael J. Cowell, P.E.

September 1998

Purpose

The Geopier™ Foundation Manual provides the geotechnical and structural engineering community with information upon which to understand basic assumptions and methodologies used in evaluating Geopier-supported structure design or Geopier-reinforced soil design, including settlement control, uplift control, lateral resistance control and global stability. All final Geopier designs must be made by Geopier Foundation Company, Inc.

This Geopier™ Foundation Manual is intended solely as information for geotechnical and structural engineers to enable them to better understand the analysis methods and assumptions employed by Geopier Foundation Company and its Licensees in designing Geopier systems. Geopier Foundation Company, Inc. and its Licensees are not responsible or liable for any designs made by others that employ Geopier technology unless such designs are approved in writing for a specific project by Geopier Foundation Company, Inc.

TABLE OF CONTENTS

FOREWORD

One of the great rewards in teaching is to witness the successes of one's former students. One of the greatest honors is to be invited to tell about it.

Nat was a Captain in the U.S. Army Corps of Engineers when he came to Iowa State University to obtain an MS degree in engineering. He wanted to work on something new and potentially important, perhaps developing some new kind of equipment, so I handed him the Borehole Shear Tester prototype, a crudely built chunk of steel concocted from a couple of automobile brake cylinders intended to predict pile skin friction. Nat took over, and with a characteristic combination of luck, hard work, and a light touch that were to become his trademark, results came out far better than anticipated. The target moved from pile skin friction to determining the drained cohesion and angle of internal friction of soils, more rapidly and with greater precision than any other field or laboratory test. As a result, the Borehole Shear Test became commercially available and is now used worldwide. Meanwhile, Nat's own target also shifted to include both MS and Ph.D. degrees, which he obtained from Iowa State in near record time. Later, he played a key role in development of a new test to measure the lateral in-situ stress in soil.

While doing his Army tour in Vietnam, Nat introduced lime stabilization that turned the supremely squishy, not to mention superlatively odorous, paddy soils into solid ground for roads and airfields. The success brought attention and recognition that, unfortunately, went mainly to Nat's superior officer, which was not all bad because it helped Nat to decide that the Army was not his career.

After Nat stepped away from a promising career in the Army, he went on to found a highly-successful consulting geotechnical engineering firm in Atlanta, Georgia. It was there that he first saw the need for an alternative to overexcavation and replacement of poor foundation soils. Thus was planted the seed that was to grow into the unique system called the Geopier intermediate foundation system.

Results from the first Geopier element experiments were again much better than expected. Bearing capacities were higher, and settlements of test piers, and subsequently of buildings sited on the new foundation system, were less than predicted by traditional methods for reasons that are only now beginning to be understood.

One key factor appears to be the high lateral stress induced in surrounding soil by pounding in the Geopier aggregate in layers. Other factors include the low compressibility of the finished Geopier and its excellent engagement with the soil. These are very strongly developed, particularly when compared with conventional concrete, wood or steel pile. The high lateral stress, low compressibility and intimate soil contact help create an unusually effective foundation system, even in relatively poor soils.

We foresee an excellent future for the Geopier foundation system, and I extend my best wishes for continued growth and success.

Richard L. Handy

Richard L. Handy, Ph.D.
Distinguished Professor Emeritus, Iowa State University
Author of *The Day the House Fell*
August 21, 1998

INTRODUCTION

Developing this manual was one of the most rewarding tasks I have undertaken in my nearly four decades of professional life. Seeing my idea grow from a hobby to a passion, and from occasional acceptance and sporadic projects to a widening acceptance and a continuous flow of new projects within the continental United States, is the epitome, to me, of the *American Dream*. This is especially rewarding within the field of foundation engineering, known more for its adherence to methods which are decades, if not centuries old, than for new technologies. Working practically every day for the past eight years on Geopier foundation design, marketing, technology improvement and construction review, has provided me with a wealth of first-hand knowledge and experience.

In graduate school, geotechnical engineers barely have time to do more than to absorb everything thrown at us. There is seldom time to question theories. We have to memorize them and try to understand their assumptions and limitations. And we accept them. Conversely, after leaving school several years to several decades ago, we are now challenged by new thoughts and new ideas that were not presented to us while we were students. Such may be the case with Geopier foundation technology. This is why I am excited to be able to provide a new technology to the design and construction industry that represents a *breakthrough in foundation engineering*.

Breakthrough

Geopier intermediate foundations can be considered a breakthrough since they are the first verifiable solution to settlement control that fills the gap between deep and shallow foundations in the United States. This technology is presently understood by less than 200 geotechnical and structural engineers in the U.S., thus the need for a Geopier Foundation Manual. Yet, in the past five years, over 200 full-scale load tests have been performed, over 180 major projects have been completed in 24 of the 50 states and in two countries. Successful projects have been performed in poor soils consisting of peats to solid waste landfills to organic fills, uncompacted fills, very soft saturated clays, and very loose sands. Equally impressive, more than half of our projects have been in fair to excellent soils--virgin soils with allowable bearing pressures of 2500 psf to 4000 psf where loads were too great for settlement control with bare shallow footings.

Not a single case exists of structure settlements exceeding design settlement using the two-zone settlement analysis method developed in 1992. In fact, our biggest challenge is to try to explain why settlements are often so much less than we estimate and predict. How can the *Peabody Place Office Building and Parking Garage* project in Memphis settle only a maximum of 1/4 inch when we designed it to settle 1-1/4 inches? We do not have complete answers. We have ample evidence that our design-build system controls settlements and differential settlements to less than designed values. We are striving to better understand the subsoil stress distributions, load-transfer mechanisms, and soil-structure interactions involved, by sponsoring university research and by utilizing special instrumentation and in-situ testing methods on selective Geopier foundation project sites.

Limitations

It is clear that no facet in the field of "earthwork engineering" is without its limitations. Historically, theories are developed and refined to provide a rational approach to solving problems. Normally, specialized sampling and testing of soils is required to properly utilize these theories, none of which are without numerous assumptions that can be either accepted or opposed. Yet, accurate solutions are normally possible only if soil strata are relatively homogeneous and extend horizontally for great distances. The cost involved for specialized sampling and testing on projects is often prohibitive. The end result is that on most subsurface exploration projects, the soils data obtained is only an approximate forecast of what exists, and it is necessary to observe the behavior of the soil during construction and modify the design accordingly. In foundation engineering, this may involve testing of subsoils at each footing location during construction. Or for deep foundations, this may involve installing probe piles or probe piers during construction, and testing selected piles and piers. This approach may become problematic since the construction cost of the outcome is unknown. Few construction projects have limitless budgets, however, given certain boundaries, these methods are often the only practical solution considering the testing constraints which often exist during the project design phase.

Nature Must Be Recognized

Subsoils change frequently and without warning for "virgin" soils formed by nature. When man gets his hands into it, with urban development and re-development, with the burying of debris and the re-filling of valleys, and undocumented placement of utilities, the situation can become pretty messy. In this, the nearing of the 21st Century, a simple fact of the geotechnical consulting profession within the United States is that cost often drives who is chosen for professional work. The scope of work, the number and depth of borings, and sampling and testing programs, are generally part of that bid package. Unfortunately, the low bidder is usually the company with the leanest testing budget. As a result, soil test borings are becoming shallower and farther apart. Hedge words and qualifications for recommendations are becoming more and more a necessity. What happens when soils change between borings? There had better be testing at each footing and redesign when appropriate. Either that, or design must be based on the worst foreseeable subsurface condition. During Geopier foundation installation, each pier can essentially constitute a test boring or a test pit. Inherent to the Geopier foundation system is the characteristic of compensating for poorer soils. Bottom bulbs naturally become larger and longer (deeper) in weaker soils. Occasionally, an unstable Geopier element will be built. It is immediately recognizable as such, re-drilled, and replaced with a good one.

Historical Sketch

Geopier short aggregate piers are nothing more than what they were intended to be initially in 1984 when I decided to *develop a refinement and practicable improvement for overexcavation and replacement.* Overexcavation, which was, and is, popular worldwide, has so many practical limitations that I felt someone should develop a better way. Geopier foundations are now routinely replacing both overexcavation and deep foundations throughout many regions of the United States. They have replaced 105 foot-deep caissons and 90-foot long piles, and are supporting 2200 kip individual column loads and 6000 kip combined column loads on single footings and strip mats. Geopier foundations are allowing economical development in areas, such as dredged alluvium fill and solid waste landfill sites, which until now have been considered uneconomical for development. Geopier foundations are also providing additional earthquake protection for shallow footings in seismic areas. Using special Aggregate Drain design, Geopier elements can provide positive protection to prevent structural damage caused by liquefaction. Geopier elements, which have internal friction angles of 52 degrees, are providing improved global stability within reinforced soil zones for remediation of active landslides and to enhance global stability to protect embankments and retaining wall projects. Geopier foundations are expanding in scope beyond providing support for commercial buildings to include support of transportation and industrial structures.

Reader Request

I ask you readers to do two things. First, *read the Manual with an open mind.* We are excited about this new technology and about the results that we see everyday. We are presently averaging two full-scale load tests somewhere in the U.S. every week. That is an annual rate of 100 load tests a year. The feedback data are voluminous and valuable. We expect, in 1998 alone, to support over 100 projects and 125 structures. By the end of this century, less than two years from now, we expect to have projects completed in at least 35 states and in half a dozen foreign countries. Why? Because the system works, because it is practical, economical and verifiable.

Second, we ask you to *make the effort to understand the system.* Many of you will find that Geopier foundation systems will be a valuable tool to add to your arsenal of foundation alternatives. There is no magic in it. There are many levels of understanding, just as there are for any solution in soil mechanics. At the most basic level, the technology is alarmingly simple and understandable. We are merely providing one or a group of piers consisting of well-controlled, very dense aggregate to replace a volume of weaker soil at selected locations to reinforce and stiffen supporting subsoils. Why does it work so well, and better than most could imagine? Some of the answers are contained in this Geopier Foundation Manual. We are actively sponsoring research and development efforts to continue our quest for knowledge. We are happy to share the knowledge gained from our research and development efforts with interested geotechnical and structural engineers to continue to improve Geopier foundation technology.

This Manual discusses a number of reasons for the excellent settlement control performance of Geopier reinforcements. On-going and future research, as well as technical developments and selective in-situ testing and instrumentation, will add to our depth of understanding of Geopier foundation technology. Our research efforts have already shown valuable technical "spin-offs," as efforts to better understand how the Geopier system works have uncovered new and potentially exciting knowledge that can be applied to settlement phenomena in non-Geopier foundation applications.

We also seek your input in developing ideas and answers. This Foundation Manual will never be a completed work, and will always be, as is the art and science of geotechnical engineering, a work in progress. We will provide errata sheets and inserts to individuals holding Geopier Foundation Manuals as we update this document.

Nathaniel S. Fox, P.E., Ph.D.
Principal Patentee and President, Geopier Foundation Company, Inc.
September 14, 1998

1.0 BACKGROUND AND HISTORY, 1984-1998

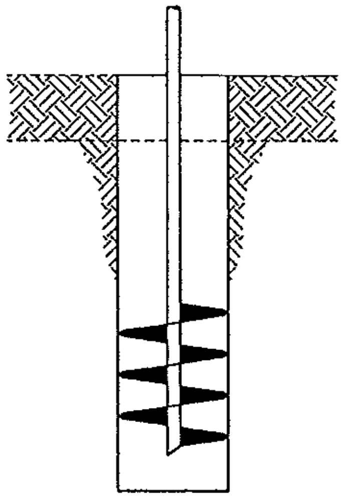
1.1 Initial Idea and Basic Concept

In the spring of 1984, Dr. Nathaniel Fox began the development of the vertical soil reinforcement method that became the Geopier Intermediate Foundation System. His objective was to refine the ancient method of improving poor and unsuitable bearing soils by removing a volume of soil and replacing it with select material of better quality. Commonly called the "overexcavation and replacement" method, it typically requires close compaction control to achieve uniformly acceptable results. The select materials used for overexcavation methods are ordinarily aggregate (stone), or high-quality soils. Limitations of this method typically include constructability, undermining of adjacent structures, uplift seepage forces associated with shallow groundwater, limitation in extent of treatment depth, , volume of replacement materials required, limitation of capacity coupled with lack of verification of capacity, large construction area affected by this method, and cost.

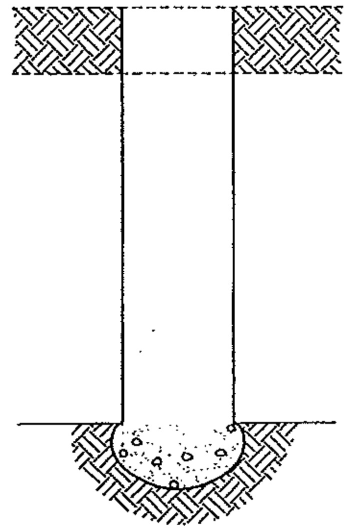
Thus, the initial objective in developing Geopier foundations was to provide a more practical and efficient process for replacing weak and compressible soils with stronger and stiffer materials - graded aggregate or granular materials, using relatively small construction equipment. Criteria included: taking maximum advantage of soils unique engineering behavior, particularly in areas of soil prestressing and prestraining; producing a higher capacity composite mass; providing a practical method of verifying capacity to control settlements of shallow foundations supported by short aggregate pier elements; and reducing the volume of select replacement materials required.

On March 21, 1984, the development of Geopier foundations began. Dr. Fox called two eminent geotechnical educators with whom he had had significant involvement in prior years, Dr. Richard L. Handy, Distinguished Professor Emeritus, Iowa State University; and Dr. Richard D. Barksdale, Professor at the Georgia Institute of Technology. Encouragement and suggestions were provided by both of these men. The concepts leading to the development of Geopier soil reinforcement and Geopier intermediate foundations were subsequently refined and expanded with the help of the co-patentee, Dr. Evert C. Lawton, Associate Professor at the University of Utah.

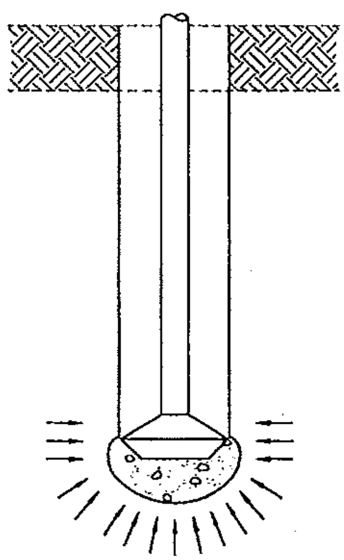
The basic concept (Figure 1.1) consists of the removal of a volume of compressible materials, either by drilling a hole or by excavating a linear prismatic volume of soil with a backhoe or similar piece of equipment. The soil at the bottom of the resulting cavity is prestressed and prestrained with an effective energy source. A very stiff element is then constructed within the cavity using well-graded aggregate placed in thin lifts and highly densified with the same energy source used for bottom prestressing. The adjacent matrix soils are improved, not primarily by densification, but rather by



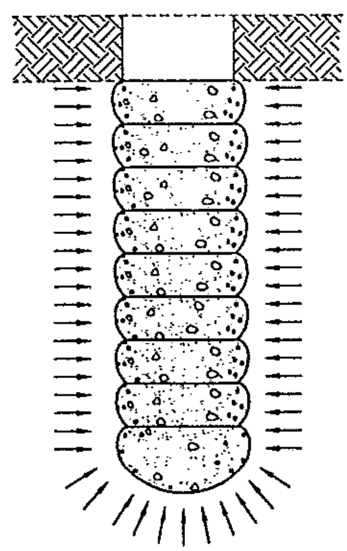
1. Make cavity



2. Place stone at bottom of cavity.



3. Make a bottom bulb. Densify and vertically prestress matrix soils beneath the bottom bulb.



4. Make undulated-sided Geopier shaft with 12-inch (or less) thick lifts. Build up lateral soil pressures in matrix soil during shaft construction. Use well-graded base course stone in Geopier element shaft above groundwater levels.

Figure 1.1 Typical Construction Process lateral matrix soil prestressing. The buildup of lateral soil stresses in the surrounding matrix soils develops an over-consolidated soil surrounding each Geopier element, resulting in a stiffened Geopier element/matrix soil mass. This lateral prestressing is maximized by use of a 45-degree beveled tamping apparatus and an impact ramming action energy source, rather than a vibratory energy source.

1.2 Testing the Basic Geopier Foundation Concept

The two co-patentees tested the basic concept of a special short aggregate pier by using drop weights placed in drilled holes, hand-held mechanical compacting equipment, and special round-headed, beveled tampers attached to modified hydraulic hammer energy sources. They found that the most practical method of densifying the aggregate and prestressing the matrix soils at the bottom of the cavities and along the shaft of the aggregate piers was to use the round-headed, beveled tampers and a modified hydraulic hammer energy source. Vibrating energy was tried and found to be less effective than the impact energy of a modified hydraulic hammer. It was also found that a hammer with a limited amplitude, high magnitude force, and a relatively high frequency of 300 to 600 cycles per minute, worked best. Rated energy of the various energy sources used ranged from 250,000 ft-lb. per minute to 1.7 million ft-lb. per minute.

Modulus load tests were performed on aggregate piers to determine their stiffness modulus values. Aspects of both footing load tests (ASTM D1194) and quick pile load tests (ASTM D1143) were incorporated into the aggregate pier load test procedures. Modulus load tests were performed to measure the improvements in stiffness which were achieved by increasing the aggregate lengths beyond a length-to-width ratio of 2.0, and to determine the magnitudes of aggregate pier modulus values that were achievable within different soils and different subsoil conditions. "Tell-tales" were installed in a series of modulus load tests to determine the approximate reduction of vertical stresses at the bottom of the aggregate piers. The tell-tales helped determine the percentage of total measured deflections that occurred as a result of aggregate compressing within the Geopier element, and the percentage of deflection that occurred from compression of subsoils below the aggregate bottom bulb.

Model aggregate pier tests were also performed in the laboratory to determine what improvements could be achieved with different select replacement materials, different spacing layouts and various length-to-width ratios of small aggregate piers. Because of inherent model-to-prototype distortions, results of the small-scale model tests were used qualitatively, and not quantitatively. Quantitative results were achieved only in full-scale field testing.

The objectives of the early aggregate pier testing program were to define practicable load test procedures and approximate aggregate pier capacities in differing subsoil conditions. Early testing also verified previously published information that length-to-width ratios of granular or aggregate piers beyond a ratio of 3.0 do not result in significantly stiffer aggregate piers within homogeneous soil strata.

Load test procedures have been slightly refined over the ten years of short aggregate pier load testing performed since 1988, and are presented in Section 5.2. Load tests are designed to provide a conservative measure of the stiffness of the tested aggregate pier. They are not intended to be long-term tests to determine long-term subsoil behavior below the "Upper Zone" stiffened by the Geopier element. They are solely for the purpose of measuring the stiffness modulus of the Geopier element. Settlement below the Geopier element is determined by conventional soil mechanics analyses. The Geopier modulus load test is a relatively quick test, since deflection of the granular aggregate pier within the Upper Zone occurs rapidly. Longer-term "creep" movements that may result from longer duration tests represent the influence of the "Lower Zone" soils and contribute to the conservative nature of the measurements from the load test.

1.3 Differences Between the Geopier Foundation System and Other Foundation Systems

Geopier intermediate foundation systems represent a new concept in foundation engineering. Historically, geotechnical engineers have had a choice between supporting building foundations on either shallow or deep foundations systems. Shallow foundations can be defined as those with a depth-to-width ratio of less than 1.0. If the underlying soils are capable of supporting the loads of a structure without excessive settlement or bearing failure, then shallow foundations can be used. However, if the soils are too weak or too compressible, or if the loads are too great, the geotechnical engineer must make a choice between overexcavation and replacement with stronger more suitable materials or transferring the loads to deeper or more suitable layers using a deep foundation system. (See Figure 1.3.1). Deep foundations can be defined as systems with a depth-to-width ratio typically between 10 and 100 (See Figure 1.3.2). Deep foundations and overexcavation can be relatively expensive foundation solutions.

Geopier intermediate foundations represent a new choice. With a typical depth-to-width ratio between 2 to 5, Geopier foundations provide an economic alternative to either overexcavation or deep foundations (See Figure 1.3.3). The combination of very stiff Geopier elements with typical allowable bearing stresses of 8,000 psf to over 20,000 psf, and matrix soil with allowable bearing pressures of less than 1,000 psf to over 4,000 psf, result in a composite Geopier element-soil matrix system with allowable bearing pressures for composite Geopier-supported footings of 3,000 psf to 10,000 psf.

1.4 Geopier Foundation Milestones

Geopier Foundation Technology has been used successfully for over 10 years to control settlements of buildings ranging from single-story commercial and industrial buildings to 16-story commercial structures, and for individual column loads up to 2200 kips each.

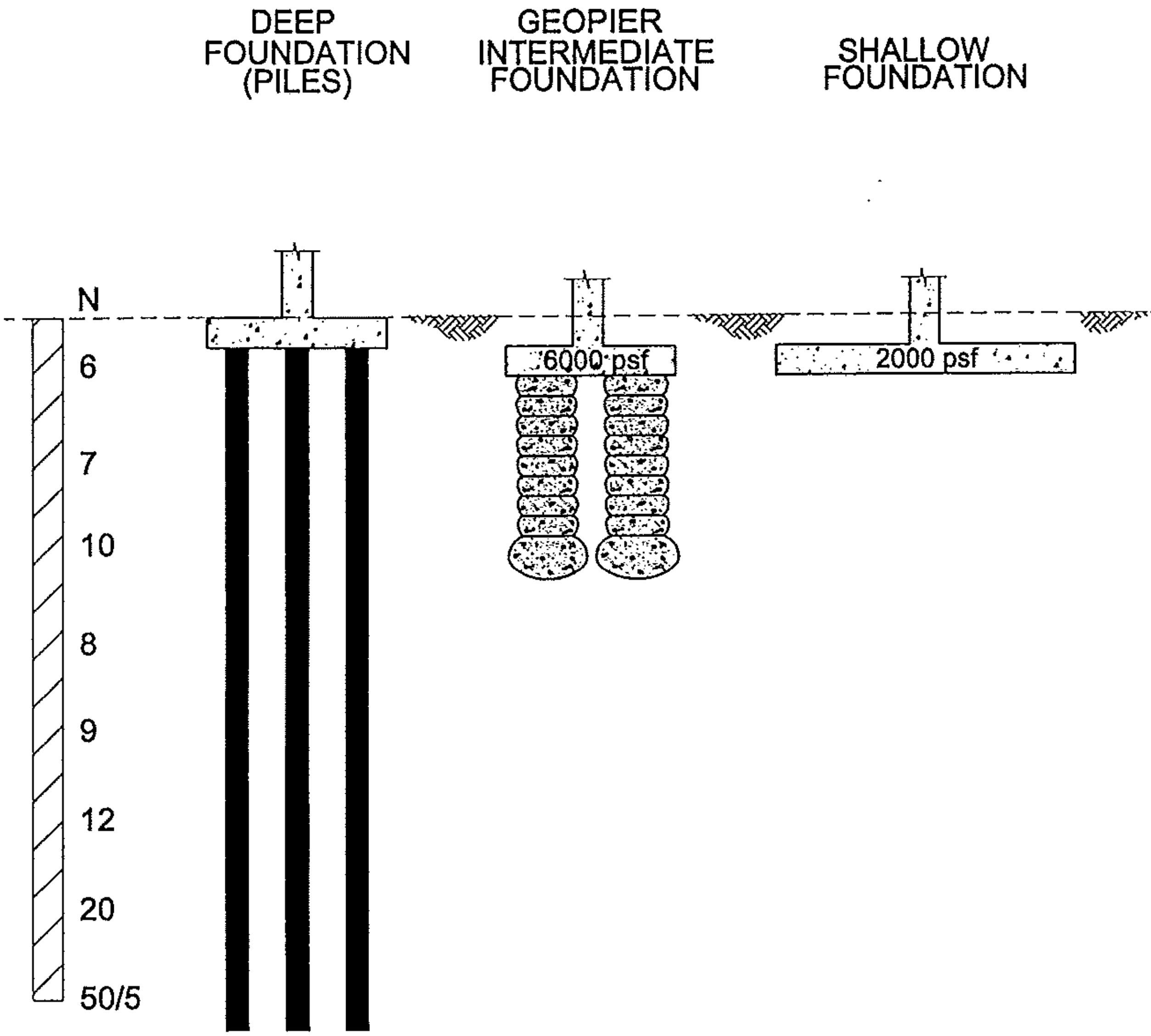


Figure 1.3.1. Foundation Alternatives in Weak Foundation Soils

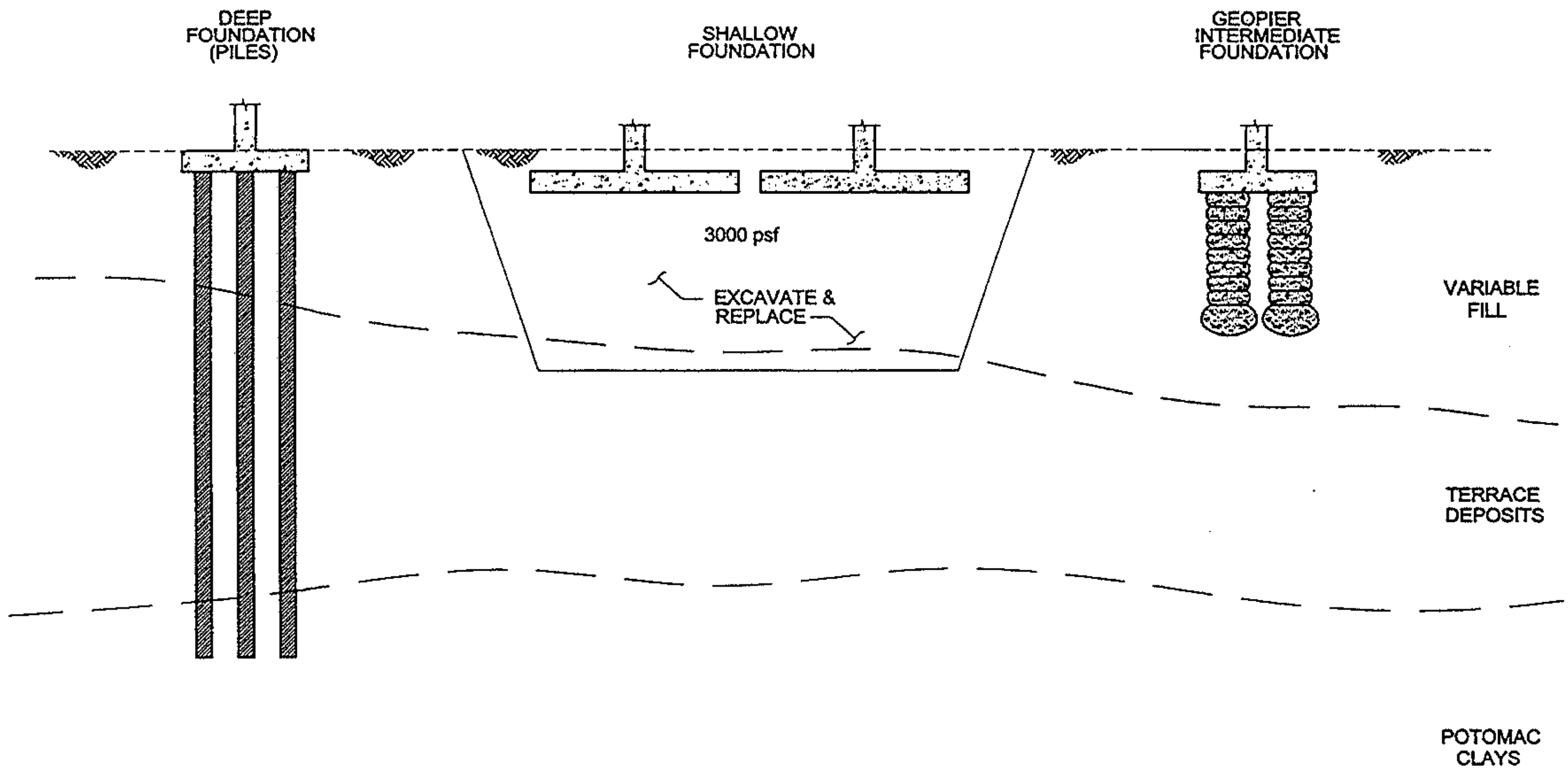


Figure 1.3.2. Foundation Alternatives in Variable Fills

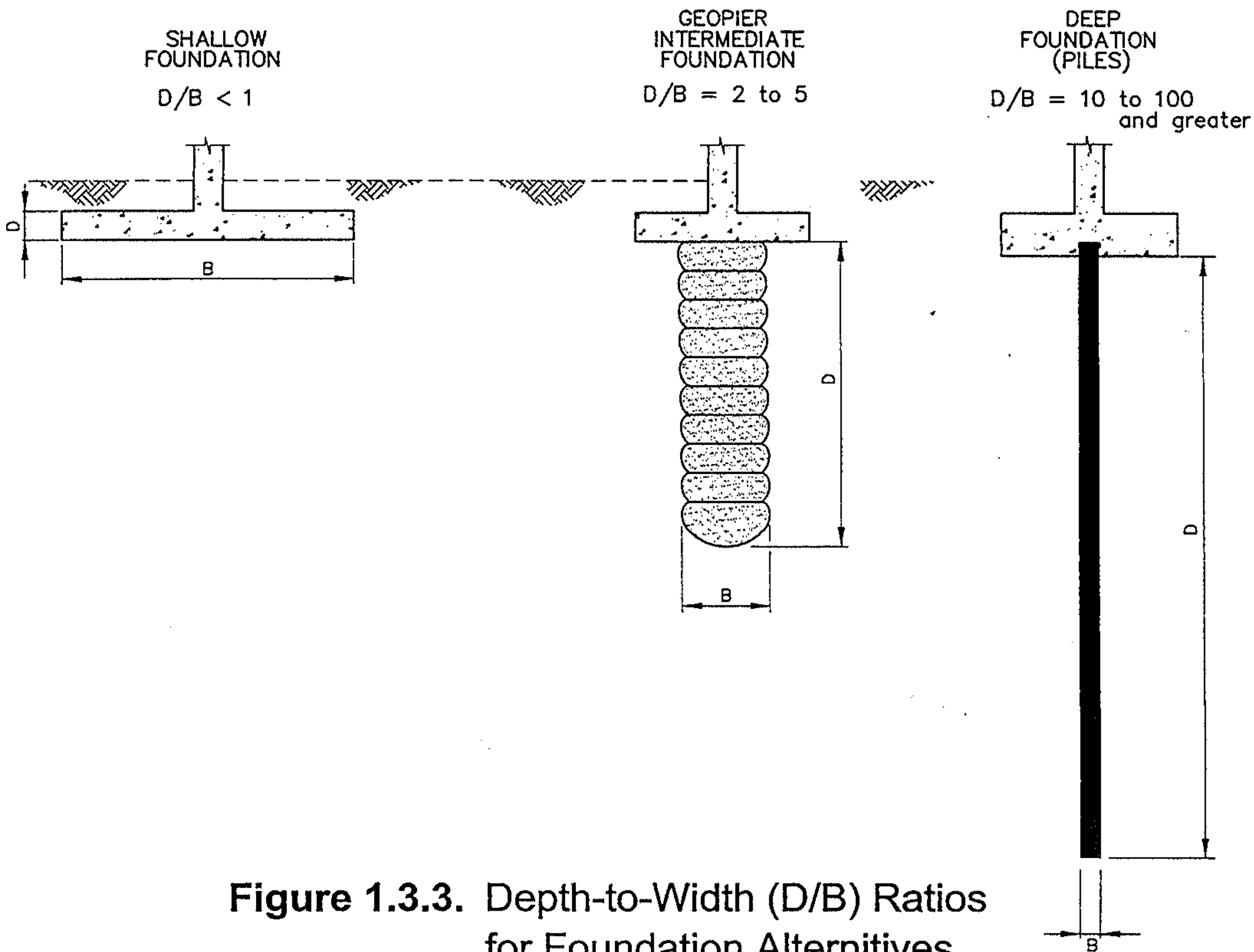


Figure 1.3.3. Depth-to-Width (D/B) Ratios for Foundation Alternitives

In addition to settlement control, Geopier reinforcements have been used to increase the slope stability for retaining walls on highway and commercial sites, to provide high capacity lateral load resistance for Geopier-supported footings and mats, and to provide uplift anchors for buildings and retaining wall structures.

The following is a summary of Geopier foundation milestones:

1988 FIRST PROTOTYPE COMMERCIAL PROJECT – Atlanta, GA

The first major commercial use of the short aggregate pier, or "Geopier" foundation process, occurred in the spring of 1988, in Atlanta, Georgia. A relatively lightly loaded, single-story, steel-framed, glass-walled botanical garden structure was designed to be built on an old organic silt fill soil. Fill thickness averaged 31 feet, and the matrix silt soil had been mixed with tree parts. Short aggregate piers, 6 feet long and 3 feet in diameter, were designed and installed to reinforce these soils and to provide positive settlement control for small footings. One hundred sixty-eight "Geopier foundations" were installed in 3 days to support 168 round footings that were placed in the same three-foot diameter hole that had been drilled for the Geopier elements. Three years after construction, maximum settlement reported was 1/4 inch.

1989 FIRST PROTOTYPE PROJECT USING SPECIALIZED TAMPER SYSTEM – Columbia, SC

The first commercial use of modern Geopier foundation installation apparatus on a major project was in December 1989, in Columbia, South Carolina. The structure was a five-story office building and auditorium that comprised a section within the city of Columbia that was nearly a full city block long. Maximum column loads were 800 kips. Soils within the site were residual micaceous silts and sands of low and moderate consistencies. The foundations were originally designed and bid as 60 to 70-foot long auger cast piles. Since the site was within a rated seismic zone, horizontal tie beams were required by building code to keep pile caps from moving horizontally as a result of the shaking of the pile and consequent lever-arm action on the caps. The use of Geopier foundation-supported footings resulted in the elimination of all the piles, as well as 85% of the horizontal tie beams. The composite Geopier element-matrix soil supported footings were designed as uniformly supported 7,000 psf allowable bearing pressure footings. Modulus load tests performed on site confirmed the design modulus values of the Geopier elements. Geopier foundations installed included 24-inch, 30-inch, and 36-inch diameter piers. Geopier element shaft lengths varied from 6 to 10 feet. Settlement surveys performed one year after the structure was completed showed maximum settlements of 1/8 inch. Due to the unexpected small settlements ("zero" measurable settlement for most columns), the final settlement survey was performed three times.

1990 GEOPIER PATENTS

The Aggregate Pier patent was applied for in 1990 and granted in the U.S. in 1993. Foreign patents were applied for in 1990 and were awarded in 1995. Principal claims, which were approved, relate primarily to the process of making an aggregate pier. This process includes making a cavity; prestressing, prestraining, and densifying the soils below the bottom of the cavity; making a bottom bulb using limited quantities of select materials; using select materials placed in relatively thin lifts to produce a very stiff aggregate pier with undulating sides; and horizontally prestressing the matrix soils at the sides of the cavity while densifying the thin lifts of select materials.

1992 FIRST MAJOR UPLIFT ANCHOR PROJECT- Meridian, MS

The first major project incorporating Geopier uplift anchors was constructed in Meridian, Mississippi in the summer of 1992. A state-of-the-art hangar with massive doors that opened like venetian blinds was built at Key Field, an Air National Guard airfield. Design uplift forces of up to 420 kips per footing were calculated to occur from wind loads. Traditional anchors presented problems of correct anchor locations within tolerances, high cost, and difficulty in providing acceptable full-scale load tests. Geopier uplift anchors provided positive solutions to these three problems and were selected for the project. Project subsoils included an upper zone of well-graded sand fill overlying a thick zone of loose clayey sand. Because of some anticipated localized cave-ins, "Geo-trench" linear Geopier elements were designed and installed rather than columnar piers.

An uplift load test was performed to 30 tons. Individual Geopier uplift anchors were designed with 20-ton capacities. A total deflection of 0.91 inches under the 30-ton load was maintained for 5 hours. One hundred percent rebound of the anchor deflection occurred upon release of load.

Six months after completion of the hangar, record winds measured at 70 miles per hour hit the airfield. The instrumented hangar showed no measurable movements, using instruments capable of measuring movements of 0.01 inches.

1992 FIRST HIGH RISE MAT SUPPORTED STRUCTURE – Atlanta, GA

A 16-story tower addition to Grady Memorial Hospital in Atlanta, Georgia was designed for drilled caisson support. Access problems occurred because the tower addition required a 22-foot basement excavation with no room for ramped access. The design was changed to a Geopier foundation-supported mat. Settlement estimates for the tower addition were on the order of 2 to 3 inches without Geopier foundation support and less than one inch with Geopier foundation support. The soil consisted of an upper zone of firm to stiff micaceous silts, underlain by a stratum of stiff to very stiff sandy silt.

The day before foundation construction began, a record 4-inch rain occurred within a 12-hour period. Exposed soils became saturated. The relatively small Geopier foundation installation equipment, consisting of wheeled skid loaders and tracked backhoes, caused the saturated silts to pump. The soil surface was stabilized using a 5-inch thick mat of open-graded stone reinforced with geogrids. This was necessary to stop the pumping and to prevent cave-ins of the Geopier foundation trenches. Because of anticipated rains and the exposed basement soils, the Geopier foundation operation was performed around the clock . Two construction teams were used, each working 11-hour shifts, with one hour of maintenance and overlap between shifts. The project was completed in less than 72 hours, just prior to additional heavy rains.

Total settlements measured after construction varied from 3/4 inch in the heavier loaded portion of the mat to between 3/8 and 5/8 inch in other areas of the mat.

1995 FIRST MAJOR FLOOR SLAB SUPPORT PROJECT – Vermillion, SD

A large industrial manufacturing plant and storage facility planned in Vermillion, South Dakota presented a problem for the developer. Relatively heavy live slab loads of 500 psf, combined with very soft clay subsoils, required expensive foundation alternatives for both foundation and slab support. The 250,000 square foot, single-story facility required either massive overexcavation or deep foundation support for foundations and slabs. The overexcavation was determined to be impractical because of shallow groundwater located within the poor soils to be removed. Deep foundation costs were estimated to be on the order of $1.5 to 2.0 million dollars. Furthermore, pile support of slabs required a more expensive slab design to prevent punching shear or to provide a structural slab. The solution reached by the project geotechnical engineer and structural engineer was Geopier soil reinforcement for foundation and slab support. The slab design was a reinforced slab-on-grade, with upper steel reinforcement in areas above Geopier reinforcements and lower steel reinforcement in areas between Geopier elements. The slab thickness was 6 inches. Grid spacing of 30-inch diameter Geopier elements was 9.0 feet on a square grid pattern.

Subsoils were so soft and compressible that trucks bringing in aggregate for Geopier foundations sank up to their axles and had to be kept off the site. Tracked equipment was used for Geopier foundation installation to prevent rutting and bogging down. Modulus load tests were performed, which confirmed the design assumptions and the available Geopier foundation modulus. Special installation procedures were developed for the load tests and adhered to for production piers. Bottom stabilization was typically achieved on the fourth lift, rather than on the bottom bulb. A total of 3,400 piers were installed in 8 weeks with a special down-hole hammer Geopier foundation tamper. Two lengths of piers were designed and installed. The slab pier shaft length was 7 feet, while the foundation pier shaft length was 8 feet. Effective lengths of Geopier elements were 2.5 feet longer, or 9.5 feet and 10.5 feet, respectively.

After the grid of Geopier foundations was installed, the treated site was found to support construction equipment, including heavy-wheeled trucks. The building was designed and constructed with tilt-up exterior walls. Total settlement of walls, columns, and slabs has been controlled to well under the one-inch design settlement criterion.

This project resulted in cost savings estimated to be in excess of $1 million and allowed the project to be constructed.

1995 FIRST MAJOR STRIP MAT SUPPORT PROJECT – Memphis, TN

This project consisted of a 15-story office building and an 8-level parking deck in Memphis, Tennessee. The subsoils on the site were typical Memphis clays of moderate consistencies. Upper portions of on-site soils contained compacted clay fills with limited quantities of construction debris. The history of foundation support during the past 80 years for heavy structures in Memphis has been primarily deep drilled piers (caissons). Lengths of caissons were typically on the order of 50 to 70 feet. The project structural engineer was familiar with Geopier foundation construction, as he had recommended and used the system on previous projects. Geopier foundation design analyses showed that the 1500 kip column loads and combined column strip mats could be safely and economically supported on a system of short aggregate Geopier foundations.

Design parameters were confirmed by results of full-scale load tests on both designed Geopier elements – 7-foot shaft length, 30-inch diameter, and 9-foot shaft length, 36-inch diameter piers. Strip mats designed for a composite allowable bearing pressure of 6000 ksf, were used typically to support four columns of 1500 kips each. Footprint coverage of Geopier foundations underneath the strip mat areas was approximately 35%.

Debris fills on the site included bricks, slab segments and footing portions. Geopier intermediate foundations were taken through the debris fill, and were extended to at least their design lengths.

Settlement surveys were performed biweekly during and after construction of the structures. No measurable settlements could be read for months after construction. Post construction total settlements 18 months after construction are less than 1/4 inch. A 14-minute Geopier foundation video (which is available upon request at no charge) was made during construction of this project. The average production rate during the project was 41 Geopier elements per day.

1996 FIRST MAJOR SOLID WASTE LANDFILL PROJECT – Hackensack, NJ

A recreation facility structure was planned for construction on an urban site in Hackensack, New Jersey. Borings showed typical clayey soils of low to moderate consistencies, with one boring showing limited amounts of debris. Subsurface conditions encountered during installation of load test Geopier foundations exposed the site as a solid waste landfill. Subsequent review of documents and interviews with neighbors revealed that the site was used as a solid waste disposal site for approximately 20 years. It had been closed as a disposal site about 10 years prior to the start of this project.

Adjustments were made in design and in costing of the project because of the solid waste landfill. A large "Lo-Drill" drilling apparatus, equipped with a core barrel for drilling through concrete, replaced conventional Geopier foundation drilling equipment on the site. Productivity estimates were revised downward, as dense solid waste materials, which would slow down production, were known to be present. Drilling and coring of 30-inch diameter Geopier foundation cavities often resulted in holes "wallowing out" to diameters of 42 inches and larger. Occasionally a Geopier element was constructed which never achieved stabilization. In these cases, the piers were re-drilled, and new, stable piers were constructed.

The average thickness of solid waste materials was 16 feet, and all Geopier elements penetrated the waste by at least two feet. However, a problem surfaced after the project was complete. The developer did not have the required construction permits, and the city would not accept the Geopier foundation system and load test results. The city and the developer reached an agreement that a geotechnical firm would be selected by the city to randomly select two production Geopier reinforcements and load test them to 150% of maximum Geopier foundation design stress. The load testing would be done using dead weights. Results of the load tests revealed 1/8 inch total settlement of one pier and 3/16 inch for the second pier, both under 150% of the total maximum Geopier foundation design stress. The foundation system was then approved by the city, and permits were issued. The structure is completed, and total settlements are less than 1/4 inch.

1997 FIRST MAJOR SLOPE STABILIZATION PROJECT – Alexandria, VA

A retaining wall ranging in height up to 20 feet was being placed over a foundation consisting of up to 20 feet of a highly plastic clay fill. The wall needed to be designed to both limit settlement and to have sufficient global stability. A segmental reinforced soil retaining wall was constructed on top of the fill foundation soils, which were reinforced with Geopier elements. The Geopier foundation extended through the fill into virgin soils. Geopier elements were located on an equilateral triangular grid pattern on 5 foot centers. The composite

shear strength of the Geopier reinforced soil was sufficient to increase the global stability factor of safety to meet project design requirements.
The frictional shear resistance of densified Geopier elements was measured in an extensive full-scale field test in order to be able to determine the Geopier element shear strength. Results showed a friction angle of 52.2 degrees for crushed base course stone densified with Geopier foundation compaction apparatus, and 48.8 degrees for open-graded #57 stone (ASTM C33).

1997 FIRST MAJOR HIGHWAY PROJECT – Clinton, MD

The first major highway project for Geopier intermediate foundations was designed and approved for the State of Maryland, to support a retaining wall on MD Route 5 in Clinton, Maryland. The project included support of cast-in- place retaining walls up to 13 feet high. Sixteen different sections of wall were investigated. Subsoil conditions varied from very soft clays to moderately strong silts and sands. The new backfill behind the walls varied in height from 5 feet to 18 feet. Design settlements for all walls were one inch maximum, with differential settlements between walls of 1/2 inch or less.

Uplift forces caused by the wall's tendency to tilt had to be resisted by uplift anchors. The Geopier foundation design incorporated 30-inch diameter uplift anchors and 36-inch diameter compression Geopier foundations. Spacing of piers depended on subsoil conditions along the 16 sections, with closer spacing and less capacity per pier within the poorest subsoils, and greatest spacing and highest capacity per Geopier foundation within the strongest subsoils. Load tests were performed in two selected areas that represented the worst subsoil conditions and a more typical condition. Geopier element lengths also varied according to subsoil conditions with the greatest length (18 foot drilled depth) within the poorest subsoil area. Sliding friction was calculated to verify that lateral load resistance provided by the compression piers alone was sufficient to prevent lateral sliding.

1998 FIRST LIQUEFACTION CONTROL, AGGREGAGE DRAIN GEOPIER PROJECT

This project is a four-story commercial structure located in a seismically-sensitive area of liquefiable soils located near the Portland International Airport in Portland, Oregon. The soils consist of unconsolidated flood plain deposits of clean sands and sandy silts which extend to depths of 70 to 100 feet. Shallow groundwater exists at a depth of 5 feet. The Oregon Department of Geology and Mineral Industries has designated the site and surrounding area as having a "*High Liquefaction Potential.*" Soil consistencies were low, with Standard Penetration blow counts (N) of 1 and 2 blows per foot in the upper 12 feet, and 6 to 8 bpf at greater depths. Settlement estimates for the structure for liquefaction-induced settlements were 12 inches to 19 inches of total settlement.

The foundation system for the building was designed as a rigid grade beam system placed on grade with a structural slab. During discussions with City geotechnical engineers and the design team, the use of Geopier soil reinforcement was selected over deep piling as a result of two factors: economics and the fact that the Geopier system could accommodate significantly larger magnitudes of lateral spread during a liquefaction event than could rigid piles.

Geopier elements extending to depths of 18 to 20 feet below the ground surface were installed at spacings of about 6 feet on centers. At column locations, clusters of up to 5 piers were installed at 4-foot centers to provide additional soil stiffness. The extremely loose soils with high groundwater, resulted in a need for the use of steel casing during Geopier installation. Casings were raised one foot for each pier lift. Special stone gradation was utilized to provide the high permeability needed for the stiff Geopier elements to also act as "Aggregate Drains" to relieve pore water pressures during an earthquake event, and to provide a reasonable and prudent approach to mitigation of the liquefaction hazard.

1.4.1 Summary of Geopier Foundation System Statistics

Geopier intermediate foundations have been used extensively in the United States for the past 10 years for applications ranging from settlement control of buildings to soil reinforcement for increasing the stability of unstable slopes. The following statistics represent the history of Geopier intermediate foundations and Geopier soil reinforcement from March, 1988 through August, 1998.

1) Geopier intermediate foundations have been installed in 24 states in the United States, and in 2 foreign countries.

2) Geopier reinforcements have controlled settlements in supporting single column loads as high as 2200 kips for individual footings, and 6000 kips for multiple columns within composite strip mats.

3) Uplift Geopier foundations are supporting airplane hangars, high-rise buildings, stadiums, and retaining walls on approximately 25 projects. Design uplift capacities of up to and including 25 tons (50 kips) per Geopier element have been achieved. Load tests have been performed to 50 tons for single 30-inch diameter and 36-inch diameter Geopier elements.

4) Geopier soil reinforcements are increasing slope stability resistance on active landslides and under retaining walls built over weak fill soils.

5) Geopier foundations are supporting structures on solid waste landfills and heavy debris-laden fill soils.

6) Geopier elements are supporting retaining walls and embankments behind

retaining walls on highway projects and building structure projects.

7) Geopier foundations are supporting structures within earthquake-sensitive areas in California and elsewhere and are providing improvement in earthquake resistance of structures through increased sliding resistance at the foundation/soil interface.

8) Geopier foundations are supporting moderately loaded structures on previously "unbuildable" sites with extremely poor subsoil conditions, including peat, very soft fine-grained soils and very loose coarse-grained soil.

9) Cased Geopier elements have been successfully installed on numerous projects in soils that would not remain open, such as in high-groundwater, sandy-soil areas.

10) Settlements have been controlled for design criteria as stringent as 1/2-inch maximum settlement for building additions.

11) Hundreds of full-scale Geopier element modulus load tests have been performed. Most of these tests have been monitored by independent Geotechnical consulting firms as part of their Quality Assurance service.

12) Full-scale direct shear tests have been performed on Geopier foundations built with both well-graded base course stone and with select, clean stone (#57 stone).

13) Geopier intermediate foundation systems have saved millions of dollars for developers of building sites by providing them with an economic alternative to conventional overexcavation and replacement and an economical alternative to traditional deep foundation systems.

14) Geopier intermediate foundations have saved General Contractors approximately 350 construction weeks on their construction schedules since 1990, and approximately 200 construction weeks within the past two years.

2.0 THEORY AND APPLICATIONS

2.1 General Theory

Geopier intermediate foundations are both a specialized foundation support system and a vertical soil reinforcement system. The unique properties of Geopier foundations are developed by the specialized construction method. Geopier foundations are constructed as follows:

- **Excavating a cavity** in the matrix soil.
- **Vertically prestraining and prestressing the matrix soil** at the bottom of the cavity while making a bottom "bulb" with selected aggregate;
- **Creating a dense, undulated-sided Geopier foundation shaft** on top of the bottom bulb by compacting thin lifts of well-graded aggregate using an impact energy source that causes a "ramming" action. Vibrating energy is not as effective in densifying the Geopier element.
- **Creating high lateral stress build up and lateral prestraining** within the matrix soils surrounding the Geopier element during the construction.

Ramming action through a relatively small amplitude, high frequency and high impact energy source not only densifies the aggregate, but also simultaneously prestrains and prestresses the matrix soil laterally. The lateral stress build-up in the matrix soil is maximized by the geometric configuration of the Geopier tamper head, with its 45-degree beveled sides. Vertical impact forces from the modified hydraulic hammers are partially converted to horizontal forces. These horizontal forces push aggregate laterally against the confined soil walls. The soil "pushes back," creating increased lateral stress in the matrix soils (See Figure 2.1.1). In-situ Stepped Blade Tests (Handy, 1997) performed adjacent to Geopier elements installed within a soft silty (loess) soil that was near saturation indicate that the pressure build-up influence extends at least 6 feet horizontally and over 4 feet below the bottom of the Geopier foundation.

Geopier foundation elements are very stiff members, as a result of the type and intensity of energy used in densification, the use of a well-graded aggregate, and the resulting confining effect of the cavity walls, enhanced by prestraining and prestressing of matrix soils which occur during the construction process. The matrix soil surrounding the Geopier elements is enhanced, not primarily by soil densification, but through build-up of vertical and lateral soil pressures. Effective prestraining and prestressing of matrix soils occurs as a result of this process of high frequency, high energy, low amplitude impact or ramming action. The rated energy of present Geopier foundation installation apparatus ranges from 250,000 ft-lb. to 1.7 million ft-lb. per minute, while the impact or ramming frequency generally varies from 300 to 600 cycles per minute. The result is an improved matrix soil that has a significantly higher preconsolidation pressure than unimproved matrix soil. This preconsolidation pressure or high lateral stress within the matrix soil makes it capable of providing increased lateral support to the Geopier element when loaded.

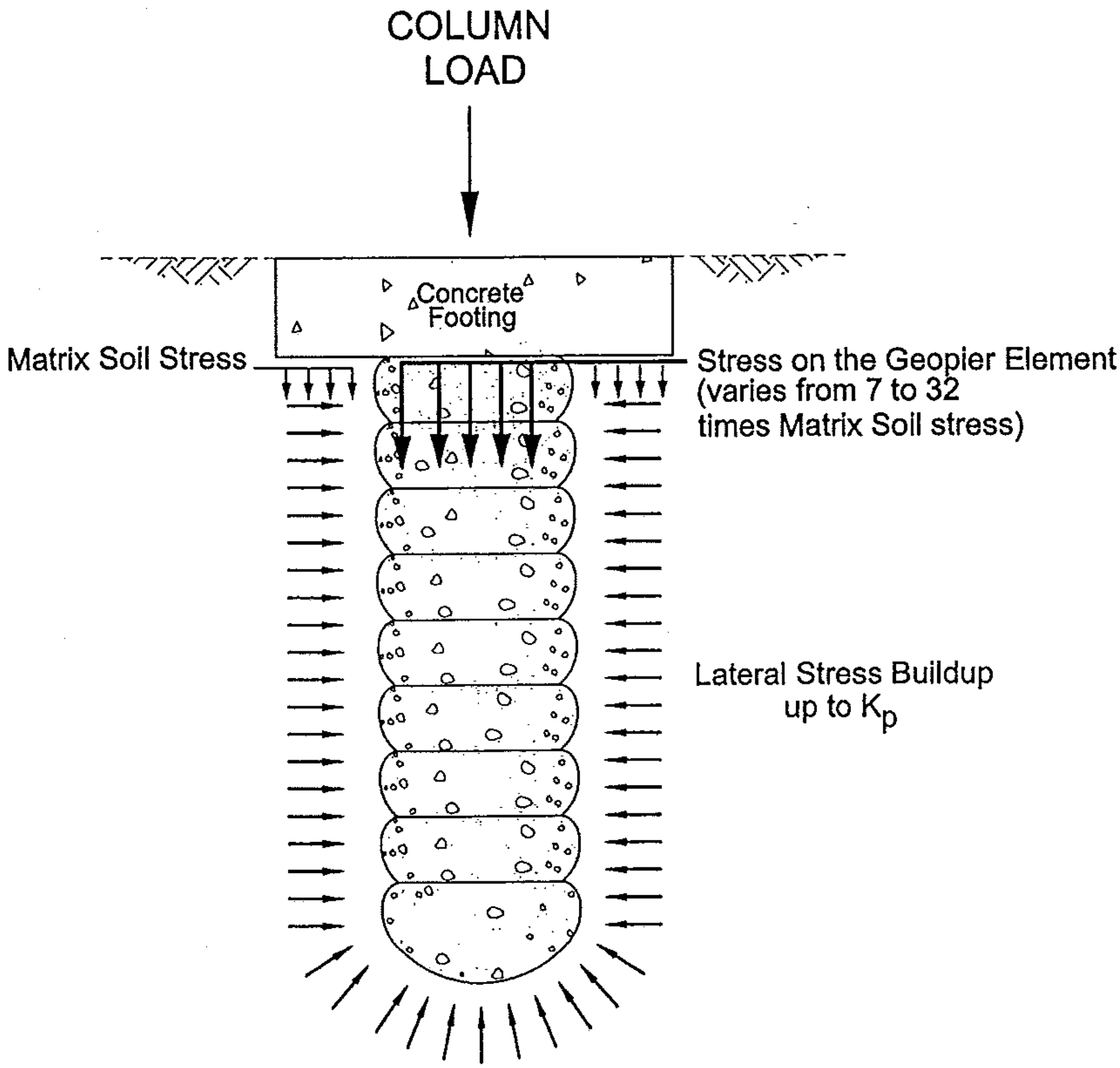


Figure 2.1.1. Geopier Load Support

As represented in Figure 2.1.2, an idealized sketch of a Geopier element, the Geopier foundation shaft is an undulated-sided column (or linear prismatic pier when Geo-trenches are made), with uneven bulges caused by the higher energy placed at the top of each thin lift. The depth of the bottom bulb and volume of soil in each side bulge is a function of the strength/stiffness of the matrix soil (Figure 2.1.3). The stronger and stiffer the matrix soil, the smaller the bulb. The weaker and more compressible the matrix soil, the larger the bulb. Bottom bulbs as long as 5 feet have been made in very soft soils, and bottom bulbs less than 6 inches in thickness have been made in relatively strong and stiff matrix soils.

Typical stiffness of short aggregate piers have been measured to be from 8 to 32 times stiffer than the stiffness of the surrounding matrix soils. A starting point in estimating Geopier foundation behavior is to assume a ratio of 10 for Geopier element stiffness to matrix soil stiffness. The Geopier element, being granular and very stiff, can be approximated as a stiff spring (Figure 2.1.4). Its behavior in compression is not elastic, however, its deflection or compression under load occurs rapidly except for possible slow consolidation creep from end bearing and Lower Zone influence. Under most loading conditions, observed from recording hundreds of full-scale load tests, rates of deflection under load become less than 0.01 inches per hour within 30 minutes of loading, except for relatively high loads or in very soft clay soils. The higher rates of deflection are influenced by consolidation of underlying soils below the bottom bulb and below the "Influence Zone" of the Geopier element.

Deflection of Geopier elements is caused primarily by three mechanisms: 1) compression of the aggregate within the aggregate pier itself; 2) vertical displacement downward as the matrix soil undergoes limited strain to mobilize perimeter shear resistance along the shaft of the Geopier element; and 3) compression and consolidation of the underlying soils within the "Lower Zone" below the Geopier element. This third mechanism is not taken into account in analyzing load test results. Ignoring this mechanism of Lower Zone settlement is one of several conservative "safety factor elements" incorporated in Geopier foundation settlement analyses. Compression within the aggregate pier and mobilization of matrix soil shear strength to provide vertical shear resistance occur rapidly and is the reason for the rapid reduction in deflection measurements during Geopier foundation modulus load tests.

"Tell-tales" installed at the bottom of Geopier elements and at the interface between the bottom and the top of the bottom bulb have consistently indicated that very little stress is felt at the bottom of the Geopier element. For length-to-width ratios as low as 2.0 and varying up to 5.0, vertical stresses at the bottom of Geopier elements have always been less than 18%, and typically less than 10% of the relatively high stress intensity at the top of the element. Recent full-scale testing of Geopier-supported footings using load cells has shown stresses on the order of 2.5% of the stress on the Geopier element at depths of four diameters (Figure 2.1.5). Stress distributions within Geopier elements will vary depending upon matrix soil characteristics and Geopier properties including stiffness.

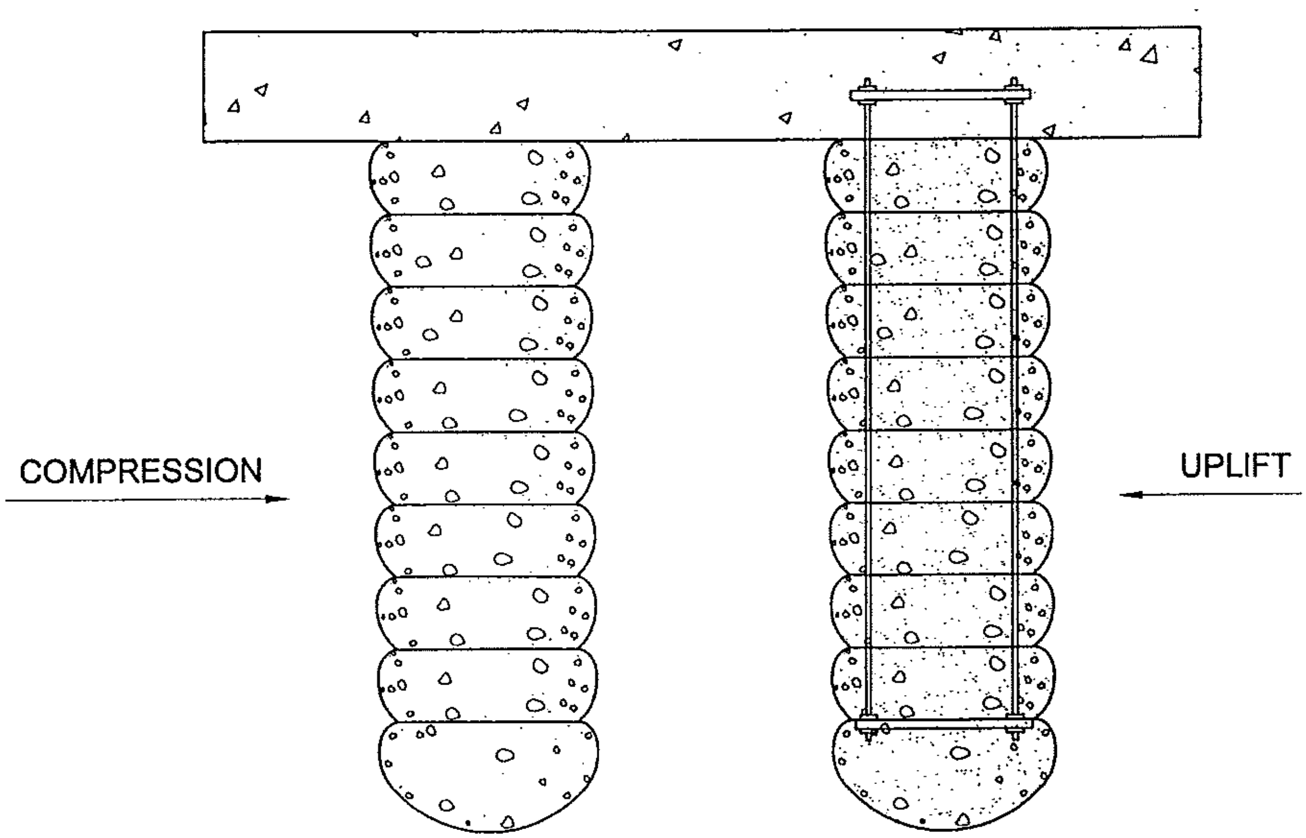


Figure 2.1.2. Typical Geopier Foundation Cross Sections

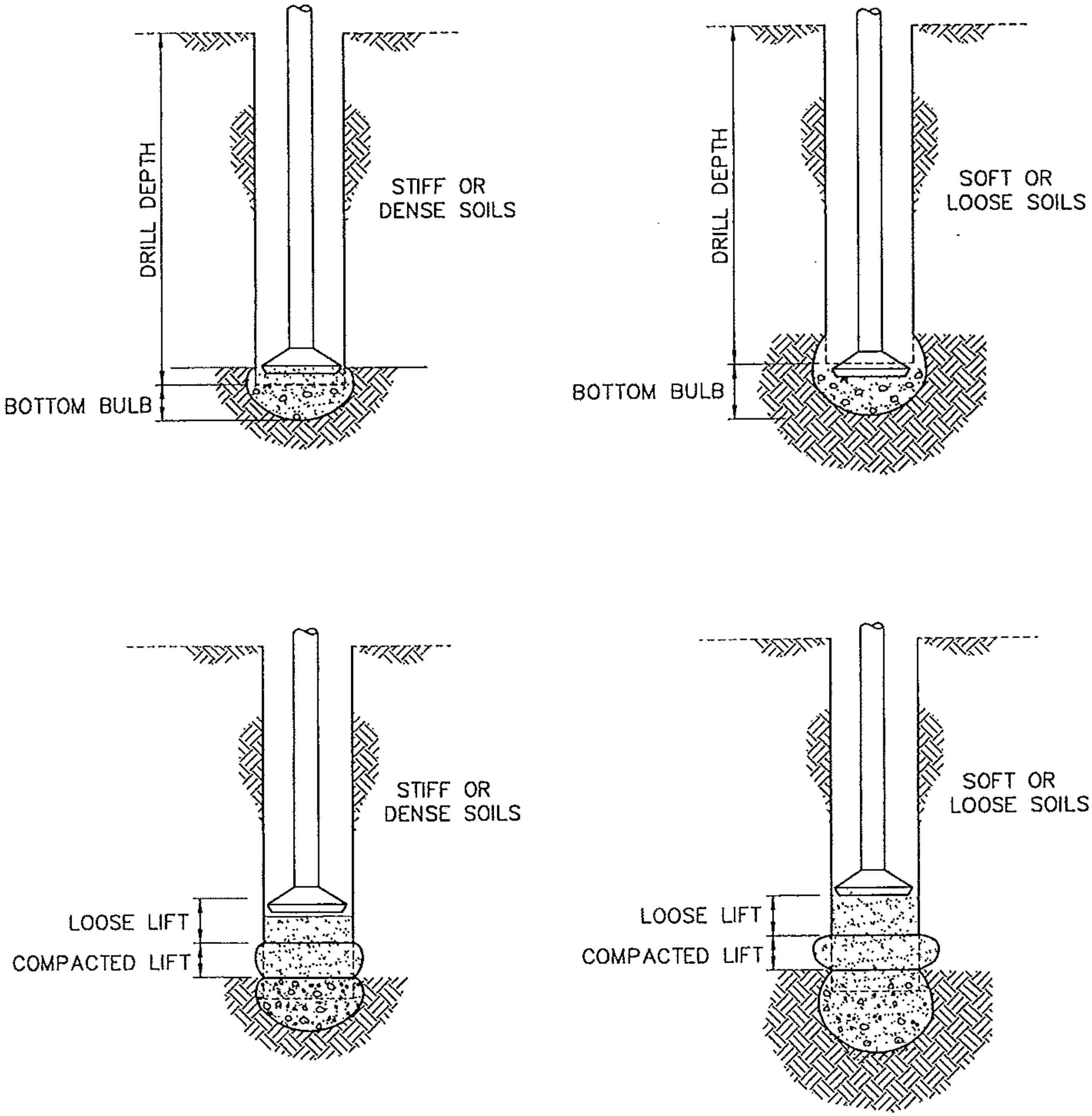
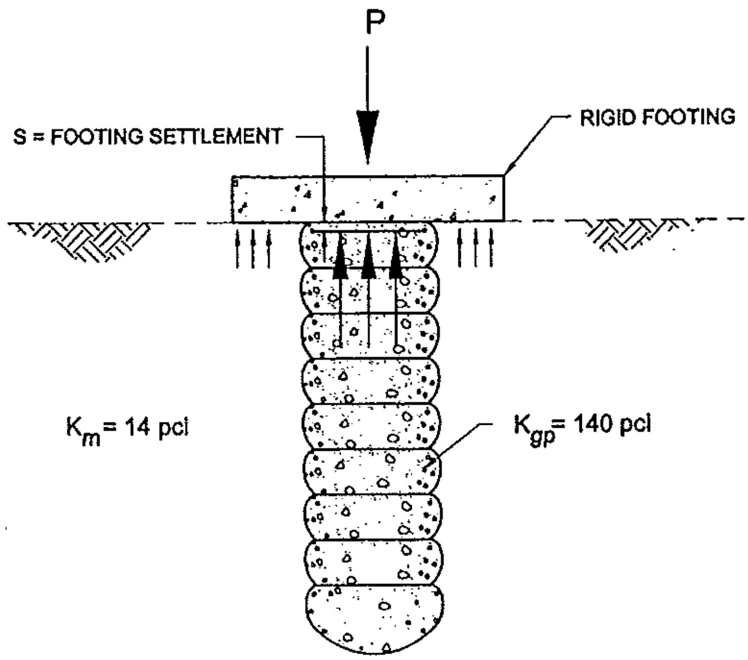
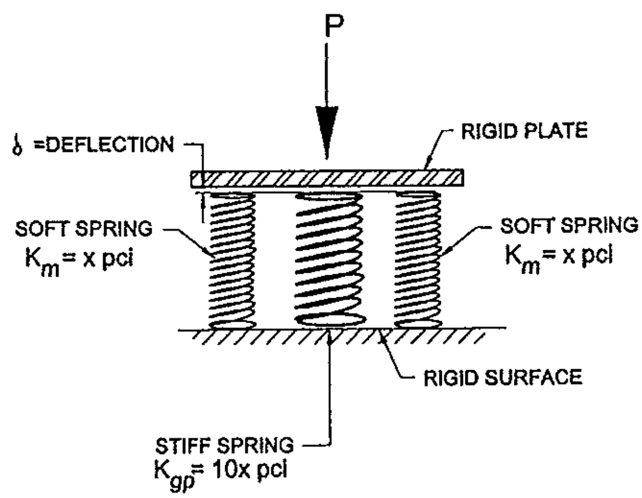


Figure 2.1.3. Effect of Matrix Soil Stiffness on Bottom Bulb Size and Lift Thickness of a Geopier Element



GEOPIER ELEMENTS AS "STRESS SINKS"



STIFF SPRING ANALOGY

Figure 2.1.4. Basic Geopier Element Theory

I-15 Northbound Bridge over South Temple, Salt Lake City, Utah
Geotechnical Test No. 2 - June 27, 1998
Distribution of Compressive Vertical Stress within Middle Geopier
1st Push Cycle at Various Lateral Displacements of Bent Cap

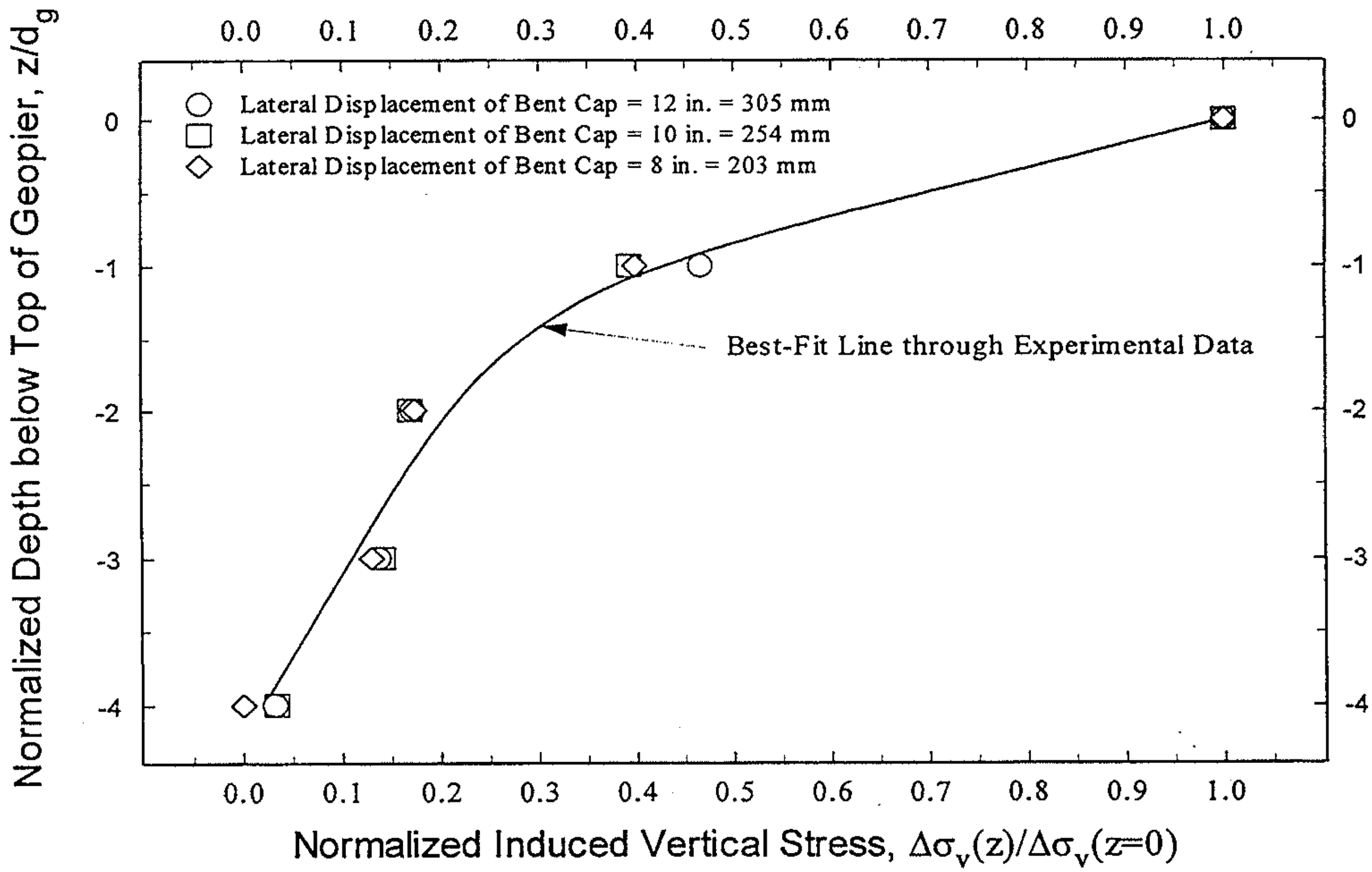


Figure 2.1.5 Stress Distribution Within Geopier Element

Since Geopier foundations may be considered to be stiff springs and the matrix soil within the Geopier Element Influence Zone may be considered softer springs, analysis of behavior of the Upper Zone can be approximated using an elastic modulus method.

Important to understanding the behavior of Geopier elements used in supporting composite footings, slabs or mats, is understanding the static equilibrium effect of stiff spring inclusions in a less stiff spring matrix (Figure 2.1.4). Assuming that a reinforced concrete isolated footing is perfectly rigid, and further assuming that the springs and matrix soils have a stiffness ratio of "N", (Geopier element stiffness (or modulus), divided by matrix soil stiffness (or modulus)), then to satisfy static equilibrium, vertical stress intensity on top of the Geopier elements must be "N" times the vertical stress intensity on top of the matrix soil. Stresses must redistribute within the footing in order that stresses concentrate on the Geopier elements, and stresses on the matrix soils are reduced.

"N" for purposes of the following discussion is considered to be 10. The Geopier elements are 10 times stiffer than the matrix soil. For this example, the vertical stress intensity on the Geopier element is 10 times that on the matrix soil. *The Geopier elements therefore act essentially as "stress sinks" or stress magnets, attracting stress and causing a reduction of stress on the matrix soil.* The greater the stress ratio (the stiffer the Geopier element is in comparison with the matrix soil), the higher will be the stress on the Geopier element, and the lower will be the stress on the matrix soil. As shown on Figure 2.1.6, if the Geopier footprint area (A_{gp}) to total footing area (A) is 33.3%, and the footing composite bearing pressure (q) is 6,000 psf, and the Geopier element-to-matrix soil stiffness ratio (Rs) is 10, then one can compute the stress on the Geopier element (q_{gp}) as follows:

GIVEN:

$q = 6000$ psf
$R_s = 10$
$A_{gp} = 0.33\,A$
$A_m = 0.67A$

Footing Stress q = Footing load (F)/Footing Area (A) **Eq. 1**

$k_{gp} / k_m = R_s$ = stiffness stress ratio **Eq. 2**

Where:
k_{gp} = Geopier element modulus
k_m = Matrix soil modulus

Geopier Element to Footing Area Ratio

$R_a = A_{gp}/A$ **Eq. 3**

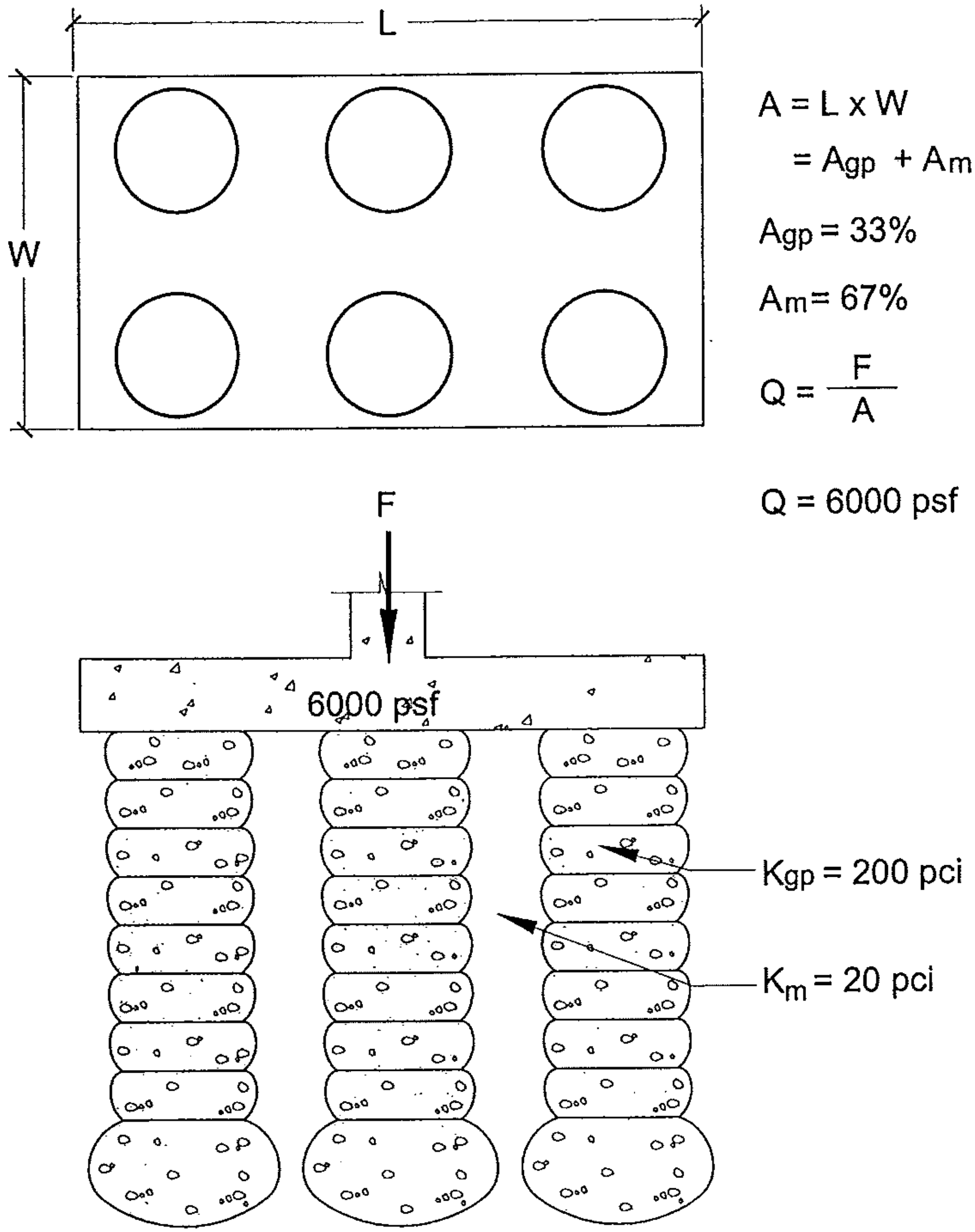


Figure 2.1.6 Typical Layout of Geopier Element under an Isolated Column Footing

Where:

A = total footing plan area
A_{gp} = Geopier element plan area under a footing
A_m = Matrix soil plan area under a footing

Rewriting Equation 1, F = qA, therefore;

$q \times A = A_{gp} \times q_{gp} + A_m \times q_m$

Dividing by footing area A results in:

$$q = \frac{A_{gp}}{A} \times q_{gp} + \frac{A_m}{A} q_m$$

Substituting Equation 3:

$A_{gp} = R_a \times A$

$q = q_{gp} \times R_a + q_m \times A_m/A$ **Eq. 4**

The area under the footing, A, can be expressed as

$A = A_{gp} + A_m$

Rewriting and Substituting Equation 3 into this results in:

$A = R_a \times A + A_m$

$A_m = A - R_a \times A = A(1-R_a)$ **Eq. 5**

Therefore, **substituting Equation 5 into Equation 4 results in:**

$q = R_a q_{gp} + q_m(1-R_a)$

Since $\frac{q_{gp}}{q_m} = \frac{k_{gp}}{k_m} = R_s$, therefore $q_{gp} = R_s q_m$

Substituting into the above Equation results in:

$q = q_m \times R_s \times R_a + q_m(1-R_a)$ **Eq. 6**

Solving for q_m

$$q_m = \frac{q}{R_s \times R_a + (1-R_a)}$$ **Eq. 7**

Substituting the given information into Equation 7 results in:

$$q_m = \frac{6000 \text{ psf}}{10(0.333) + (1 - 0.33)}$$

$\mathbf{q_m = 1500}$ **psf**

Since $R_s = 10$, then $q_{gp} = 10 \times q_m$

$\mathbf{q_{gp} = 15{,}000}$ **psf**

The stiff Geopier elements have reduced the footing stress on the matrix soils from 6,000 psf to 1,500 psf, or to 25% of the footing stress intensity. Conversely, the Geopier elements have attracted 15,000 psf, or 250% of the footing stress intensity.

For a column load of 500 kips, the total load supported by the Geopier foundations would be approximately 15,000 x 0.33 x 500,000 /6000 = 416,250 lbs., versus the total load supported by matrix soils being 1,500 x .67 x 500,000/6000 = 83,750 lbs. The Geopier foundations support 5 times more load than the matrix soil, and the matrix soil supports only 16.75% of the total footing load. If the ratio of Geopier element-to-matrix soil stiffness is 15, the characteristic of the stiff pier to attract stresses and act as a "stress sink" is even more pronounced, and the stress on the Geopier element is 16,014 psf, and the matrix soil stress is 1,067 psf. For this example, the Geopier elements support 7.5 times as much of the total footing load as the matrix soils, and the matrix soils support only 13.5% of the total footing load.

Recent full-scale Geopier foundation instrumentation, including placement of load cells within Geopier elements and within upper portions of matrix soils immediately in contact with the bottom of a rigid (4-foot thick) footing, showed the effectiveness of Geopier elements acting as "stress sinks." Stress ratios of Geopier element vertical stresses to matrix soil vertical stresses in three different test locations were measured to be 16:1, 20:1 and 32:1. This research was performed as part of the I-15 bridge seismic study in Salt Lake City, Utah.

The Geopier element is designed and built as an efficient stress resistance element. When the Geopier element is stressed, it tends to bulge slightly. The bulging causes further lateral pushing against the confining matrix soil, which in turn causes more build-up of lateral soil pressures within the matrix soil and a higher normal stress perpendicular to the vertical perimeter shear stress. This in turn increases the perimeter shear resistance provided by the Geopier element.

As with any foundation system, the allowable bearing pressure of a Geopier system is determined based on two considerations - ultimate bearing capacity and tolerable settlement. Like most footings, the allowable bearing pressure of a footing supported by Geopier elements is governed in most instances by tolerable settlement, rather than bearing capacity. This is even more pronounced with footings supported by Geopier elements than with normal soil-supported footings. This is due primarily to the significant strengthening effect of the aggregate pier within the Geopier Element Influence Zone, or the "Upper Zone." A bearing failure, if it occurred, would have to be caused either by an extreme bulging of the pier, or it would have to occur at a level below the Upper Zone, with one exception being if a very dense or very hard subsoil existed within the Lower Zone soils. An exceptionally weak soil, such as a peat, might fail by lateral bulging before excessive settlement were achieved, but even this possibility is remote.

A unique aspect of Geopier foundation system behavior is that through lateral stress impact at the interface of the Geopier element and the adjacent matrix soils, significant lateral (horizontal) stresses are induced prior to application of structural loadings. These high lateral pressures in the matrix soils "confine" the Geopier columns, increasing the shearing resistance along the Geopier element-matrix soil interface. At the same time, the more compressible matrix soils are prestrained by the application of the stresses during installation. The end result is a stiffer matrix soil material than that which would exist without the induced lateral pressures.

The bottom bulb created during installation of the Geopier element provides an end-bearing support that allows high densification of the Geopier shaft materials. The ramming action and high intensity impact stresses during construction of the bulb are maximized by using open-graded stone, which provides more efficient stress transfer by grain-to-grain contact of the larger stone particles to the underlying soils supporting the bulb. It is both intuitive, and has been observed, that the weaker the subsoils, the larger the resulting bulb of the Geopier element

The Geopier installation method, in effect, tests the soil at each lift in each Geopier element. The softer the soil, the larger the diameter of the Geopier element and the deeper the Geopier element influence becomes. Therefore, the ramming action of installation automatically compensates, at least in part, for unanticipated changes in the soil conditions at each individual pier location. In locations where a large bearing surface is required due to weak soils, a larger bulb is created.

Geopier foundations develop their ultimate bearing resistance as a combination of shearing resistance along the interface and bearing resistance at the bottom bulb of the Geopier element. Some minor bulging may occur near the top of Geopier elements. This bulging increases the area of the Geopier shaft and actually increases the bearing capacity of the Geopier system.

As previously mentioned, tolerable settlement, rather than ultimate bearing capacity, typically controls the allowable bearing pressure of a Geopier foundation system. A detailed description of the ultimate bearing capacity for Geopier foundations is provided in Section 4.3.

Settlement Analyses

Settlement analysis of a Geopier foundation is a complex soil-structure interaction problem, consisting of the interaction between the footing and Geopier element, the footing and matrix soil, and the Geopier element and matrix soil. Geopier Foundation Company has used several methods to estimate settlements of Geopier foundation systems. The most straightforward and practical method and that which has been used on every Geopier foundation project since 1992, is to separate the volume of subsoil affected by stressed composite footings into two zones, an Upper Zone and a Lower Zone. The Upper Zone or "Geopier Element Influence Zone" is considered to include the vertical distance from the footing bottom to an elevation equal to the drilled depth (or cavity excavation depth), plus a length equal to one diameter of the Geopier element. The diameter of the Geopier element is added to account for the bottom bulb and the significant vertical prestraining and prestressing which occurs below the Geopier element shaft during the construction of a Geopier element (Figure 2.1.7 a and b).

The settlement analysis method includes two basic steps. First is an analysis of settlement contribution within composite materials of the Upper Zone. Second is an analysis of settlement contribution from the Lower Zone. Settlement contributions from the Upper Zone and the Lower Zone are then added together to produce estimates of total settlement. A third step may be performed between the two analysis methods described. Vertical stress at the interface of the Upper Zone and the Lower Zone may be estimated and inserted in order to more accurately estimate the settlement contribution from the Lower Zone.

Upper Zone

A finite Grid method based on Beam-on-Elastic Foundation Theory is used to estimate the settlement component in the Upper Zone. Using this method, settlements are determined for a given load using a linear analysis and appropriate elastic moduli for the Geopier elements and the matrix soils. Modulus values of the matrix soils are calculated as described in Section 4.2. Modulus values of Geopier elements are initially estimated based on past experience from the results of hundreds of full-scale modulus load tests. Conservative modulus values for Geopier elements used in design are then determined by a modulus load test on site. Approximate values of Geopier element moduli for preliminary design analyses are provided in Section 4.2.

Interface between Upper Zone and Lower Zone

Determination of the magnitude of the vertical stress at the interface between the Upper and Lower Zones is complex. It is apparent from monitoring the settlement performance of thousands of footings supported by Geopier elements, stripmats and mats, that vertical stresses at the interface are reduced from what they would be without Geopier reinforcements. At least three conditions are believed to contribute to the reduction of vertical stresses at the interface. These are:

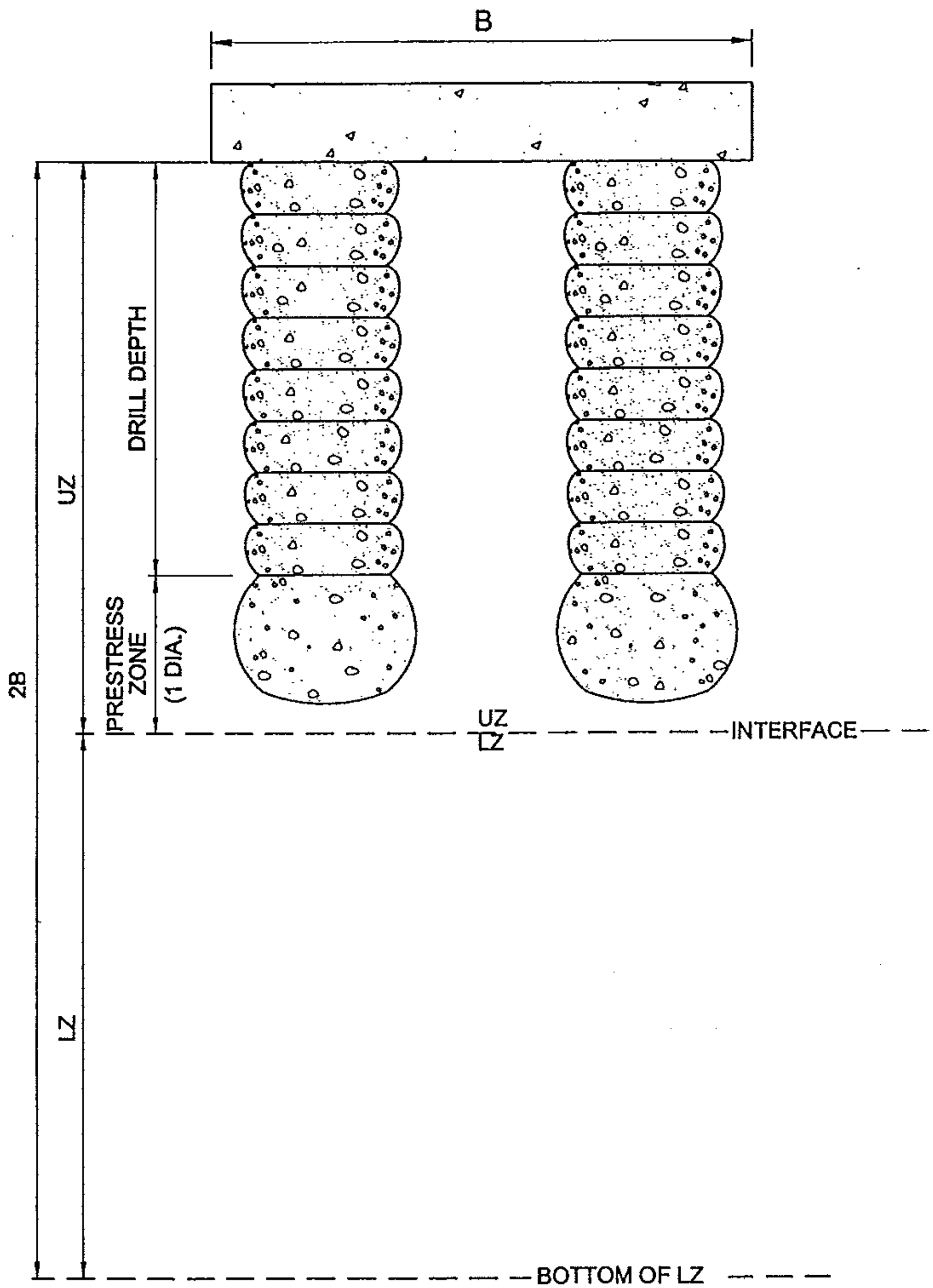


Figure 2.1.7a. Upper Zone and Lower Zone for an Isolated Column Footing

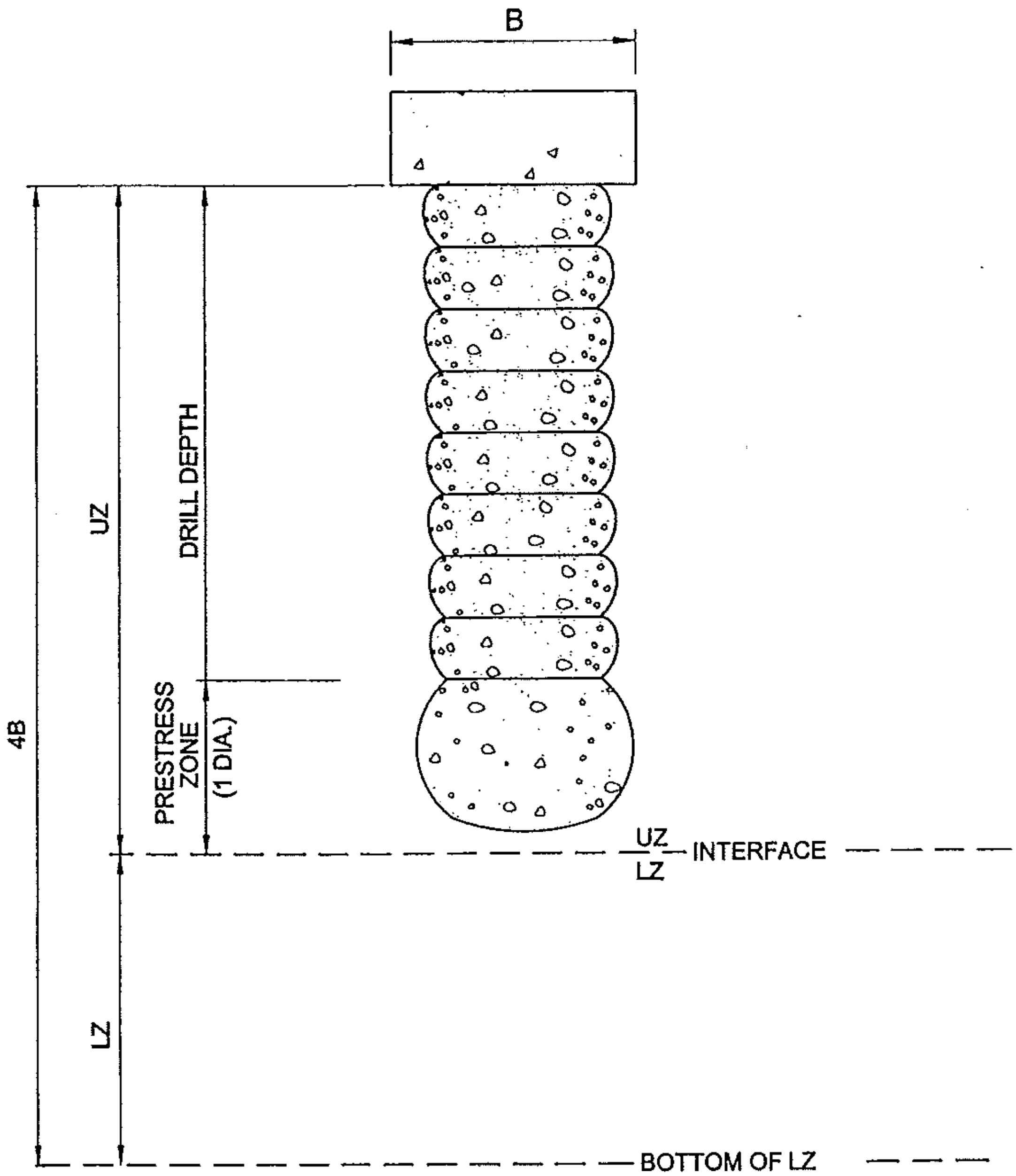


Figure 2.1.7b. Upper Zone and Lower Zone for a Continuous Strip Footing

1) Creation of a stiffer upper layer (Upper Zone) overlying a less stiff lower layer (Lower-Zone) through installation of very stiff Geopier elements and creation of stiffer matrix soils from the build-up of lateral stresses;

2) Construction of the bottom bulb creating vertical prestressing and prestraining of subsoils supporting the bulb; and

3) A dissipation of vertical stresses within the Geopier element, which results in reducing the depth to which equal stresses would extend within underlying matrix soils without Geopier elements (Figure 2.1.8).

Lower Zone

Lower Zone settlement contribution can be evaluated using any acceptable conventional soil mechanics approach. Because of the lack of available consolidation data on many projects, a layer strain method of settlement estimation is often used. The primary method used is a modified use of Schmertmann's layer strain method (Schmertmann 1975). Because of the inherent over-conservative nature of this analysis, a correction factor of 0.67 multiplied by the resulting settlement contribution is applied. This reduction factor is commonly used by geotechnical consultants in the State of Florida, who frequently use the Schmertmann layer strain method for settlement predictions and estimates. In addition to the Schmertmann layer strain method, other methods of estimating Lower Zone settlement are utilized, including consolidation theory and various elastic layer methods.

Total Settlement Estimate

The total settlement estimate for a Geopier foundation is the sum of the Upper Zone settlement contribution from compression of the composite Geopier element/matrix soil zone and the Lower Zone settlement contribution from consolidation or compression of the soil in the Lower Zone.

2.2 APPLICATIONS FOR GEOPIER INTERMEDIATE FOUNDATIONS FOR SETTLEMENT CONTROL

Currently, the major use of Geopier intermediate foundations has been to replace the need for overexcavation and replacement ("over-ex") and to replace the need for deep foundations *for settlement control of building foundations.* The following sections describe the potential benefits from using Geopier foundations in lieu of overexcavation in fill soils and in lieu of deep foundation systems when weak or compressible soils are encountered.

2.2.1 Use of Geopier Intermediate Foundations in Fill Soils

When variable fill soils are encountered at a building site, the most common approach is to remove and replace soils of "unknown origin." Removal of poor and unsuitable

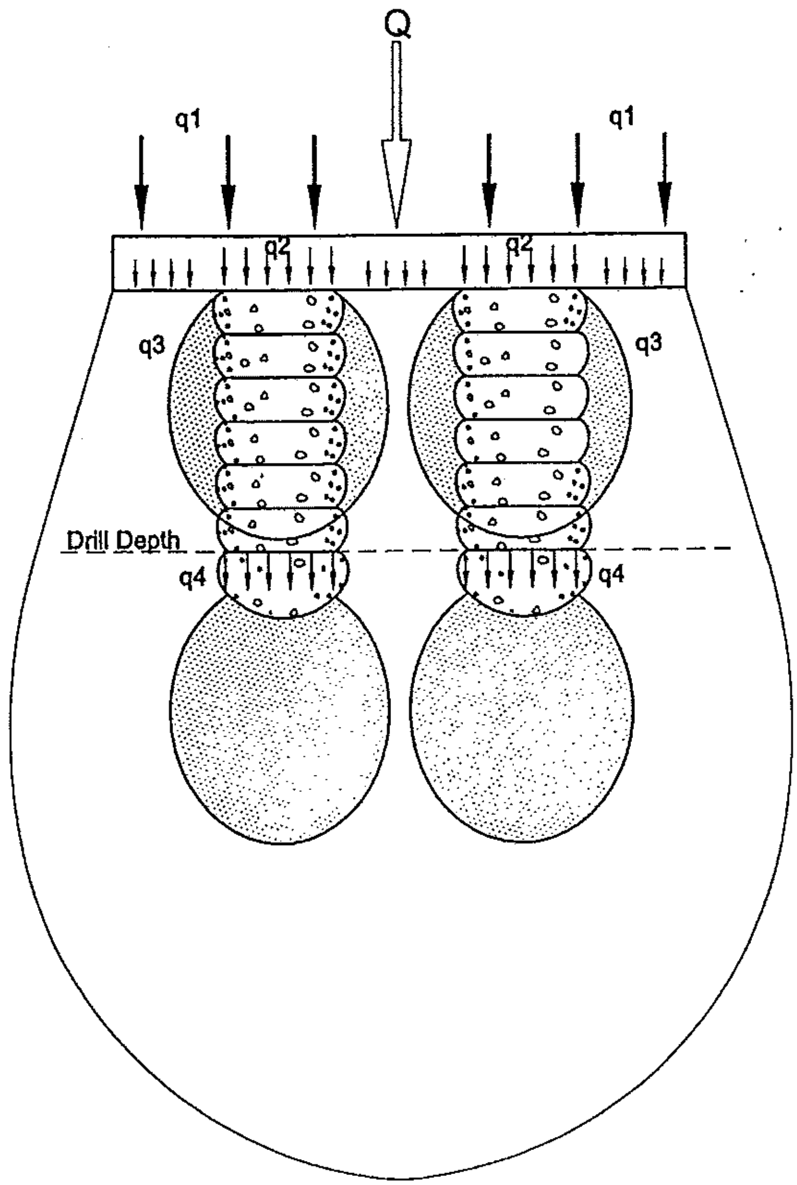


FIGURE 2.1.8 Schematic Diagram of Pressure Isobars Below a Footing Supported by Geopier Elements

soils and replacement with better quality materials, particularly granular materials, is a remedy that has been in use for centuries. It remains practical today for situations where a relatively-thin layer of poor soil exists below footing and slab bottoms, and for situations where adjacent buildings supported on shallow foundations cannot be undermined. Geopier foundations provide an alternate method to replace overexcavation, which can provide significant benefits in terms of performance and cost. Factors to consider when using Geopier foundations in fill and other poor soil applications include:

1) **Depth of excavation**. When excavation depths exceed approximately 5 feet, Geopier foundation systems are generally less expensive, safer, and less weather-dependent than overexcavation.

2) **Densification** - Even well controlled densification for a compacted backfill will not produce the density and stiffness that is produced within a Geopier element. This is because of the high level of energy applied and the confinement obtained within the confines of a drilled hole (or excavated linear cavity). Fill soils reinforced with Geopier elements will typically provide 2 to 4 times the allowable bearing pressure available from well-compacted select fill soils. This increased bearing pressure can result in smaller footings and less settlement.

3) **Variability of fill or of the poor subsoils** - When subsoil or groundwater conditions change during overexcavation, the ability to excavate and replace can be impaired and can result in increased costs. In addition, the ability to effectively compact soils becomes limited with depth. This limitation becomes even more critical when groundwater is encountered and when the stability of cut slopes in soft soils comes into question. Geopier foundations can adjust to these changing conditions more readily. As soils become softer, the bulbs of the Geopier elements naturally increase in size to stabilize poor soil areas. Likewise, as water is encountered, construction of Geopier elements can be adjusted through the use of appropriate aggregates and casing, as needed.

4) **Existing Buildings Adjacent to New Construction** – Overexcavation may not be feasible or is cost prohibitive when used adjacent to existing footings or slabs. The potential to undermine existing footings may require expensive and difficult underpinning methods to construct. Since excavations for Geopier elements are relatively small diameter holes, these can be excavated immediately adjacent to existing structures without disturbing existing foundations. The high frequency impact energy (300 to 600 cycles per minute) does not cause resonant action in soils, as does pile-driving, since soils have a natural low frequency.

5) **Ability to verify capacity** - Geopier foundation installations are routinely accompanied by full-scale load tests to verify design assumptions. Load tests are seldom, if ever, performed for overexcavation sites. As a result, settlement control is more predictable and reliable with a Geopier foundation system.

6) **Cost.** If overexcavation depths are greater than or equal to about 5 feet, Geopier foundation support is generally less expensive and less likely to result in any unforeseen costs or problems than overexcavation and replacement. Over-excavation has the potential for additional costs associated with changing conditions, the type of material (hazardous or non-harzardous) which may be encountered, and groundwater.

7) **Weather influences during construction.** These influences impact over-excavation and replacement to a significant degree, while impacting Geopier foundation installation to a far lesser degree. Rain affects moisture content of select soils brought in to replace the excavated soils, and grading work cannot be accomplished in the rain. Geopier foundation installation frequently takes place in rain or in freezing weather, conditions that would prohibit construction using the overexcavation and replacement method.

8) **Depth of excavation.** Most projects in variable fill require overexcavation of all the variable fill soils. This is because the influence depth for footings placed on poor or variable fills have a low bearing pressure, and hence, will affect a large depth, i.e., 2 times the footing width for square footings and 4 times the footing width for continuous footings. For sites with high loads and 10 to 30 feet of fill, this becomes a significant expense. Since Geopier elements act as stress sinks, they can dissipate stress over a shorter depth. Likewise, the higher allowable bearing pressures associated with Geopier foundations result in smaller footing sizes and shallower influence depths. Therefore, Geopier elements seldom need to penetrate the fill soils to control settlements. The combination of stress dissipation and smaller footing sizes associated with higher allowable bearing pressures normally results in relatively short Geopier elements, even in deep fill soils.

9) **Capacity.** Overexcavation and replacement, when properly performed, typically results in footings with 3000 to 4000 psf allowable bearing pressure, and loads on footings supported by overexcavation are often limited to approximately 200 kips. For Geopiers, allowable bearing pressures of footings typically vary from 4500 to 7000 psf, and maximum column loads often exceed 1000 kips. Even in poor subsoil areas, maximum column loads which may be safely supported normally exceed 400 kips.

2.2.2 Geopier Intermediate Foundations as a Replacement for Deep Foundations and Ground Improvement Methods

When a project has relatively high loads or when a project has relatively light loads and highly compressible foundation soils, deep foundations or ground improvement methods are traditionally recommended. This recommendation is usually made after settlement analyses performed for conventional spread footings result in excessive settlements or very large and expensive footings. For heavily loaded structures a mat foundation may be considered prior to recommending a more expensive deep foundation or ground improvement system, if settlements of the mat can be tolerated.

In taking a closer look at the settlement analysis for a shallow spread footing foundation (Figure 2.2.2.1), a problem often arises. Reduction of footing bearing pressure results in a footing with larger dimensions. This larger footing, while reducing the magnitude of the stress in the upper soils, transmits significant stress on soils for a greater depth (typically significant stresses are assumed to extend to depth equal to 2 times the footing width for column footings and 4 times the footing width for continuous footings). There may become a point where the depth of compressible soils is too great and the subsoil stress it too high at which a larger shallow footing will not perform adequately. The example shown in Figure 2.2.2.1 illustrates a case where excessive settlements are estimated using 3125 psf design bearing pressure. By reducing the bearing pressure to 2000 psf (a reduction in stress of 36%), settlement estimates remain excessive, and bare shallow footings shown are not acceptable. Reduction in settlement is only 16% with a 36% stress reduction because of the deeper stress influence with the larger footing.

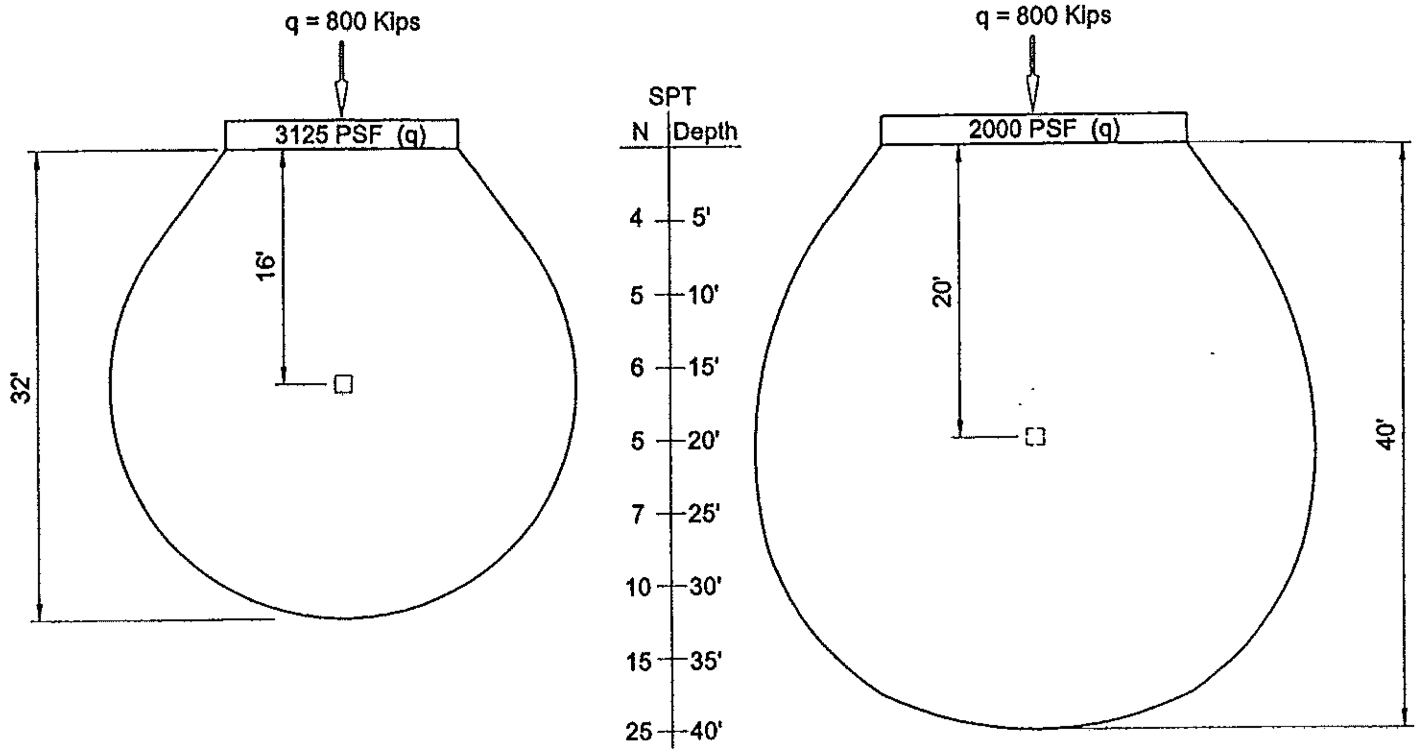
This is the time at which a deep foundation is normally introduced. However, Geopier elements used to support shallow foundation systems provide an **Intermediate Foundation System** that can usually solve this problem more economically (Figure 2.2.2.2).

Geopier elements can increase the allowable bearing pressure by 200% to 500% in compressible or loose soils. This higher bearing pressure results in a smaller footing area, and hence, a shallower influence depth for the imposed stress. Likewise, the incorporation of Geopier elements within a depth typically equal to the footing width, or less for isolated spread footings, controls settlement within that zone and reduces the stress on the compressible soils below the Geopier element. The result is a smaller footing founded on an improved foundation soil that will meet the project performance requirements for settlement.

The Geopier Intermediate Foundation System is economical versus most deep foundation systems such as H-piles, drilled shafts (caissons), auger cast piles and timber piles, when the length of pile for an equivalent system is 20 to 25 feet or more.

When compared with soil improvement methods,Geopiers elements are often economical since they generally provide a greater load carrying capacity, and a higher modulus versus other methods. Geopier foundations are often confused with stone columns (vibro-replacement), since they are usually columnar, and they both consist of aggregates placed in the ground to improve the soil. However, the two technologies are very different in terms of construction methodology , load transfer mechanisms and the degree of subsoil improvement achievable.

Stone columns utilize a large vibrating probe lowered by a crane. Weight and vibration, sometimes aided by water or air jets, cause a cavity to be made by the vibrating and oscillating motion of the probe. Soil is displaced laterally while it is forced to occupy space that had belonged to the unreinforced matrix soil. The end product is



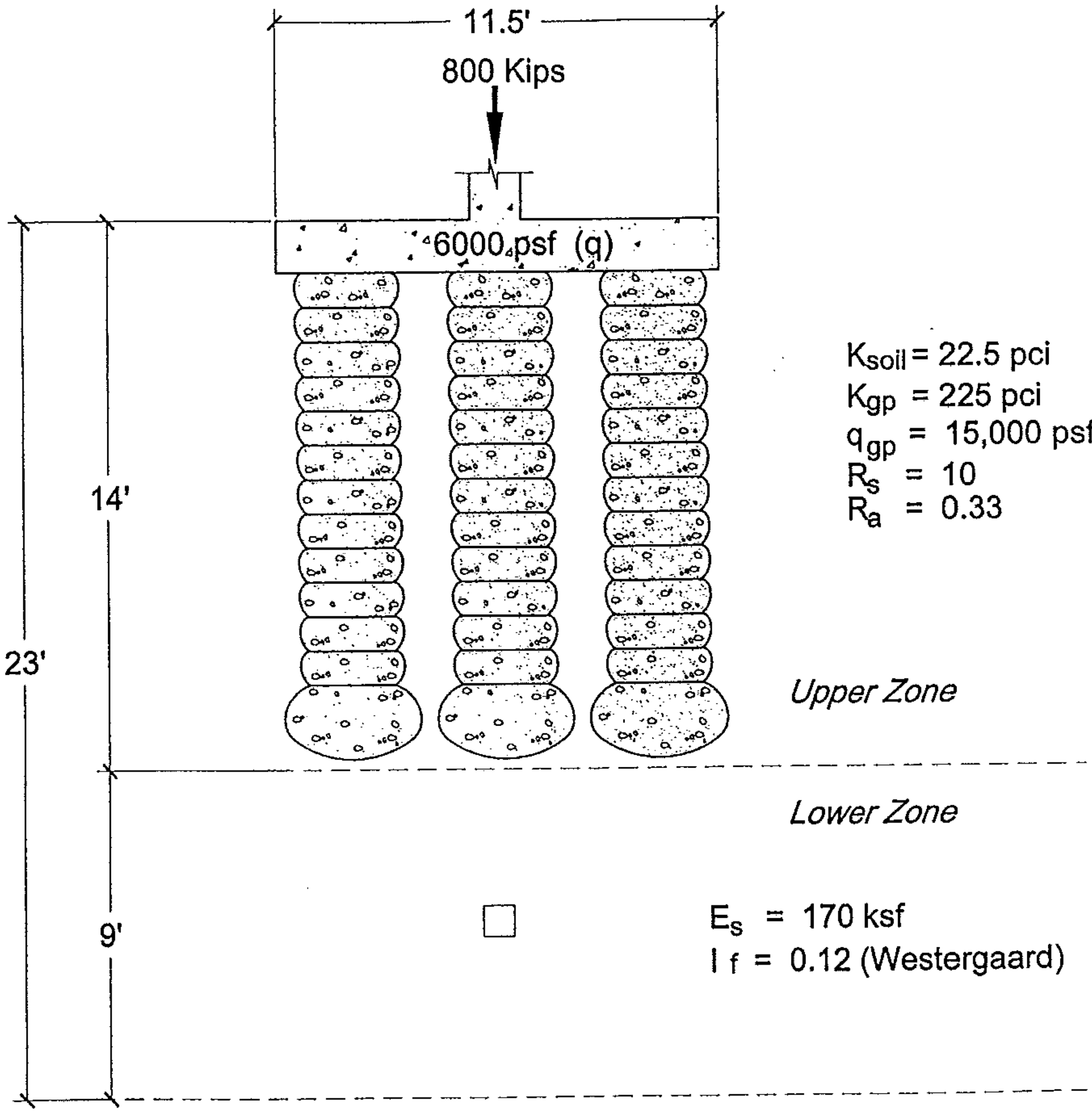
N_{avg} = 5
$E_{s\ avg}$ = 170 ksf
I_f = 0.35 (Bousinesq)
(For Homogenous Soils)

$S = \frac{q \times I_f}{E_s}$ x Thickness = 2.47 inches

N_{avg} = 5
$E_{s\ avg}$ = 170 ksf
I_f = 0.35 (Bousinesq)
(For Homogenous Soils)

$S = \frac{q \times I_f}{E_s}$ x Thickness = 1.98 inches

FIGURE 2.2.2.1 Preliminary Settlement Estimate



$$\text{Upper Zone } S = \frac{q_{gp}}{K_{gp}} = 0.46 \text{ inches}$$

$$\text{Lower Zone } S = \frac{q \times I_f}{E_s} = 0.51 \text{ inches}$$

Total Settlement = 0.97 inches

Figure 2.2.2.2 Geopier Foundation Solution a stone column with a weakened "mudded zone, " typically approximately 6 inches thick around the periphery of the column. The overcompacted and sheared soil zone beyond the mudded zone contains soils with significant excess pore pressures in fine-grained soils which may require as long as 6 months or one year to dissipate. As a result, soils treated with this method are often not stabilized for a significant time period after treatment. Shear strength of the aggregate has typically been considered to be 33 degrees because of mixing of the matrix soils (often soft clays) with the stone, fracturing of the matrix soil structure, and creation of the "mudded zones." Stiffness of stone column-treated areas has often been in the range of 2 to 3 times original subsoil stiffness.

A Geopier element is created by removing a volume of compressible soil and replacing it with a well-graded aggregate in thin lifts using impact ramming energy. This method creates large bottom bulbs and very high lateral stress within the matrix soil. The result is a stiff aggregate pier that has a significantly greater modulus and shear strength than a traditional stone column as a result of the creation of high lateral stress within the adjacent matrix soil, and the use of well-graded, highway base course aggregate with a very low void ratio, compacted in thin lifts.

2.3 Present Limitations of Geopier Soil Reinforcement

As in any foundation system and any soil reinforcement system, Geopier foundations are not the appropriate solution for all situations. There are limitations caused by site and subsurface conditions that affect the constructability of installing Geopier elements. There are also subsoil conditions which are either inappropriate or are limiting, such that the Geopier element may not effectively provide the needed soil improvement with respect to the magnitude of the structural loads. Finally, there are economic limitations that exist where other reliable systems may prove to be less expensive than a Geopier foundation system or where Geopier foundations cannot show a significant cost savings compared to alternatives. Each of these general situations are briefly discussed below:

2.3.1 Subsoil Condition Limitations.

Geopier elements have been successfully used to support individual columns with loads of up to 2,200 kips, strip mats with combined columns totaling 6,000 kips, and single mats with total loads of 25,000 kips. Geopier foundations are also supporting light to moderate loads in exceptionally weak soils, including peat soils and solid waste landfills. However, there are practical performance limitations for Geopier foundations, as with any other foundation or soil reinforcement system, and these limitations are a function of the nature of the soil being reinforced and the structural loads being applied.

Settlement analyses for structures supported by Geopier elements normally assume *homogeneous subsoil conditions*, whether the subsoils are weak, strong, or in-between. As a result, pier elements are normally designed to terminate in the "homogeneous" subsoil, and are seldom designed to extend into a better subsoil stratum. Occasionally, a very poor subsoil exists which must be penetrated by the Geopier element, such as peat soils, highly organic soils or heavy debris-laden fills. Also, occasionally a weak soil is underlain by a strong soil where the depth to the better soil can be reached easily and economically by the Geopier element without appreciably increasing its length.

There are, however, several subsoil conditions that may severely limit the soil improvement derived from Geopier reinforcements. These include very soft peat, muck, and highly organic soils, fine-grained soils with negative pore water pressure, and highly expansive clayey soils. Geopier reinforcements have been successfully installed within soils that fall into all of these categories. However, such conditions present limitations in performance, as well as limitations imposed by economics.

Peat

Peat soils, very high moisture content organic soils, and landfill materials must be penetrated by the Geopier element. The consolidation characteristics of these types of soils are such that leaving a zone of highly compressible peat, muck, or highly organic soils below the upper "Geopier-reinforced zone" could present an unacceptable risk of future excessive settlement. In general, if these soils are not deeper than about 20 feet, Geopier elements may be extended through them to better soils, and the vertical soil reinforcement provided can prove to be economically attractive, as well as providing positive settlement control. The limitation in capacity of Geopier systems taken through peat can be expected to be from bulging of the Geopier element within the peat.

Soft Clays that Develop Negative Pore Pressures

Soft clays that generate negative pore water pressures present the exception to the general rule that Geopier foundations gain strength from the time they are initially installed. In these soft clays, the strength of the Geopier element /matrix soil system can be expected to decrease within a short period of time after the Geopier element is installed. The capacity of Geopier foundations to provide settlement control is reduced when matrix soils develop negative pore water pressures. As is discussed in Section 5.3.3.3, a simple field test has been developed to determine when Geopier foundations are being installed in negative pore water pressure clays.

Weak Soils

For the general case of weak soils, including soft clays, soft silts, and very loose sands, Geopier soil reinforcement is effective and results in a significant improvement in Upper Zone stiffness. Often the composite stiffness of the Upper Zone with Geopier elements installed in weak soils may be increased 500% or more by the inclusion of a Geopier foundation system. Even with this degree of improvement, the support capacity of the Geopier foundation will be limited to light to moderately heavy structures. The capacity will vary and can be estimated by performing settlement and bearing capacity analyses for specific site conditions. Full-scale load tests will confirm estimated design parameters. Capacity varies depending on how soft the soil conditions are, the type of soils being reinforced, and whether underlying soils are preconsolidated. If poor subsoils are deep, the Geopier shaft lengths will typically be increased in order to reduce the Lower Zone thickness, thereby reducing the intensity of vertical stresses reaching the compressible soils in the Lower Zone and reducing the thickness of the Lower Zone used in the settlement analysis. This determination is readily seen when performing a settlement analysis. In general, very weak soils 30 to 40 feet deep or greater with loads above 700 kips are often better suited for pile foundations. The same soils with isolated footing widths 12 feet or less are often technically and economically feasible Geopier foundation projects.

Very weak soil layers (weight of hammer to N = 2 bpf) located at a depth of 2 to 4 B below the footing can also create problems if the footing loads are high, especially when located above a very stiff or dense soil layer. Even though the stresses are very low, these highly compressible layers may influence the settlement of buildings. This can be the case when the footings are heavily loaded and spaced fairly close together such that the Geopier reinforced soil acts more like a flexible mat foundation. In this case, the total load of the building and the modulus of each of the soil layers should be considered in assessing building settlement estimates.

Expansive Clays

Geopier reinforcements must be carefully designed when high volume change (swelling) soils are being reinforced. Some precautions and design methods employed include: 1) lowering the Geopier element so that top of the element is below the "active zone", so that volume change does not occur within matrix soils adjacent to the Geopier element shaft; 2) installing an uplift system so that the lower portion of the Geopier element resists uplift forces acting on the upper portion of the Geopier element and on the footing; 3) reducing side friction on the upper portion of the Geopier element within the active zone, either by casing the Geopier element with a cardboard or plastic columnar form, or by coating the active zone cavity walls that reduces side friction; and 4) providing a void beneath the footing in the matrix soil area so that there is room for expansion and shrinkage. One important aspect of Geopier foundation technology that contributes to the effectiveness of using Geopier foundations in high volume change soils is the concentration of stresses within a Geopier element. The stress concentration on the piers result in higher intensity vertical load and vertical stresses. This factor adds to the ability of the Geopier element to resist uplift forces that are created when moisture contents increase within expansive matrix soils. Another positive aspect of Geopier foundation systems in high volume change soils is that composite bearing pressures of the footings supported by the Geopier elements are relatively high, and typically on the order of 5000 to 7000 psf in fair to good, fine-grained soils, including high volume change soils.

2.3.2 Constructability Limitations. Geopier foundations require excavation of a cavity prior to building a Geopier element. Since most Geopier elements are relatively short, this is a relatively quick process. Likewise, once the hole is excavated for a Geopier element, construction occurs quite rapidly, i.e., 10 to 30 minutes is typical for constructing a Geopier element. There are several conditions that slow down the

process of constructing Geopier elements. Examples include high ground water associated with sands, rubble fills, and unstable soils.
The solutions to these problems center around the use of more powerful drilling equipment to penetrate rubble fills, the use of casing to control soil caving, the use of open-graded aggregate for compaction below the water table, and the selective use of pumping to control groundwater. Each of these techniques has an associated cost in terms of production time and each is discussed in Section 5.0, Geopier Foundation Construction.

2.3.3 Economic Limitations. Economic limitations of Geopier foundation systems may be taken care of in the marketplace. However, for planning purposes, it is advantageous to understand general factors that affect the economics of using a Geopier foundation. Approximately 75% of technically feasible Geopier foundation projects to date have proven to be economically feasible and significantly less expensive than alternative systems. Estimated costs in using Geopier foundations for the remaining projects show less than 25% savings and in some cases, no cost savings. We often consider that significant savings should be at least a 25% cost reduction in comparison with alternate systems. The reduction in construction time, the ability to construct Geopier foundations during rains and in freezing weather, the elimination of the potential for overruns due to changed conditions, the reduction in concrete footings volumes, and prevention of site degradation may be added to actual subcontract cost savings to provide a more accurate measure of total advantages realized by the Geopier foundation system. In such cases, a 10% subcontract bid savings may result in true savings of 25% or higher.

When rock or very dense/very hard zones are more than 25 or 30 feet deep, Geopier foundations generally provide the highest cost savings. Geopier foundations have replaced the need for 105-foot deep drilled piers and for 100-foot deep piles. Needless to say, cost savings were great in these cases. When overexcavation is the alternative to a Geopier foundation system, savings are generally achieved when the fill thickness is greater than approximately 5 feet.

3.0 GEOPIER AND MATRIX SOIL PROPERTIES

3.1 Geotechnical Information

To develop a complete Geopier foundation design, it is important to have knowledge of the properties of the soil in which the Geopier element is founded, and the soil below the element. Therefore, knowledge of the soil stratigraphy to a depth that exceeds the influence depth for the application is necessary. For designs associated with settlement, uplift or sliding of building foundations, knowledge of the soil conditions to a depth greater than 3 times isolated column footing widths and greater than 5 times continuous strip footing widths are needed. For slope stability designs, information to a depth on the order of 2 times the proposed embankment width or beyond known weak layers that will influence stability, is necessary. The soil properties that are important within these influence depths are application-specific and are discussed below.

Geopier-Supported Footing Capacity

To develop an initial estimate of the load-carrying capacity of a Geopier element and its representative footing segment, the allowable composite footing bearing pressure of a Geopier-reinforced foundation, and the stiffness modulus of a Geopier element, it is necessary to know what the consistency of the soil is throughout the depth of soil in which the Geopier element will be constructed, and to a depth of about 5 feet deeper than the cavity drill depth. For most applications the pier drill depth will vary between 10 and 18 feet. The Geopier element/matrix soil capacity, allowable composite footing bearing pressure and the modulus values of the Geopier element can be estimated from knowledge of the Standard Penetration Resistance (N-values) using Tables 4.2. Likewise, it is necessary to know the allowable bearing pressure for the matrix soil to determine the matrix soil modulus and its contribution to the support of a footing. The allowable bearing pressure of the matrix soil is often provided the subsurface exploration report and can also be estimated from N values. This value is then converted into the matrix soil modulus, k_m, in pounds per cubic inch by dividing the allowable soil bearing pressure in pounds per square inch by 1 inch of settlement.

Caution should be used on the indiscriminant use of SPT data. Cathead & rope used for imparting the load during the Standard Penetration Test, which is being phased out by some geotechnical consultants, usually is assumed to give about a 60% efficiency, and is the basis for most data interpretations. The automatic trip hammer can give upwards of 95% efficiency, and blow counts can be increased to equivalent N60 values by multiplying by the energy ratio.

Settlement Analysis

In order to perform a settlement analysis (Figure 3.1.1a) for a Geopier-supported structure, knowledge of the compressibility characteristics of the soils below the Geopier element is required. The elastic modulus of the soil can be estimated based upon

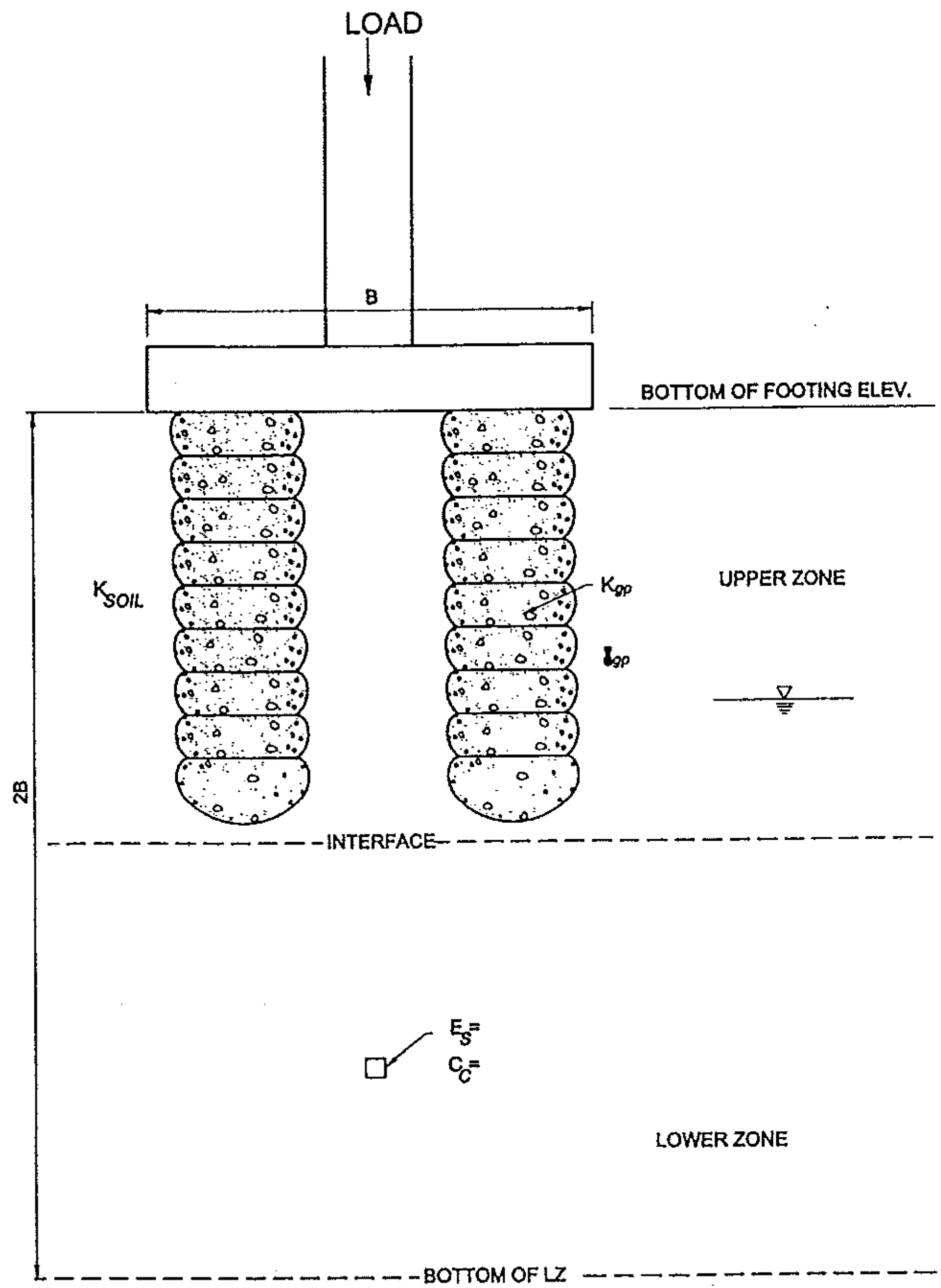


Figure 3.1.1a. Information Required to Design Geopier Elements for Settlement Control

Standard Penetration Test results (N values) (Bowles, 1977, Martin, 1987) and from cone penetration results (Schmertmann, 1975). The compressibility characteristics of cohesive soils can be measured directly from consolidation testing. Knowledge of the compression characteristics of the soil is required for soil directly below the Geopier element to a depth below the footing bottom of at least 3 times the footing width for isolated square column footings and at least 5 times the footing width (or Geopier element diameter, whichever is greater) for strip footings.

Uplift Capacity

To perform uplift capacity analyses, knowledge of the coefficient of passive earth pressure, K_p, is needed to estimate the normal stress on the Geopier element/matrix soil interface. The effective friction angle of the matrix soil is needed to compute K_p, and is also needed to determine the coefficient of friction for shear resistance of the pier. The effective friction angle may be estimated from available knowledge of the matrix soils, or measured in laboratory triaxial or direct shear tests. It can also be measured in-situ using the Borehole Shear (BST) test, which provides separate C and ϕ data. Uplift load tests, using a maximum load of 150% of the design uplift load, are used to verify analyses or to provide design uplift capacities. Maximum design uplift capacity is normally taken as the results of successful uplift load tests divided by 1.5. If uplift load tests produce a "failure," then the load causing failure is divided by 2.0 to provide maximum design uplift capacity.

Lateral Load Resistance

To be able to perform lateral sliding analyses (Figure 3.1.1b) for a Geopier foundation supported structures, knowledge of the friction angle of the Geopier element is required. Results of full scale shear testing performed on Geopier elements are provided in Section 3.2.3. In addition, stiffness ratios of Geopier element stiffness to matrix soil stiffness are required to calculate stress concentration and vertical stress from deadload on Geopier elements. This, in turn, requires modulus information on Geopier foundation elements and on matrix soil.

Slope Stability

To be able to perform slope stability analyses (Figure 3.1.1c) using Geopier soil reinforcements, knowledge of the friction angle of both the matrix soil and the Geopier element is required. Typically, drained direct shear tests or CIU triaxial shear tests with pore pressure measurements are performed on the matrix soil to obtain values for long term slope stability analyses. For short-term slope stability analyses, knowledge of the undrained shear strength from undrained triaxial shear strength testing of the matrix soil is also required.

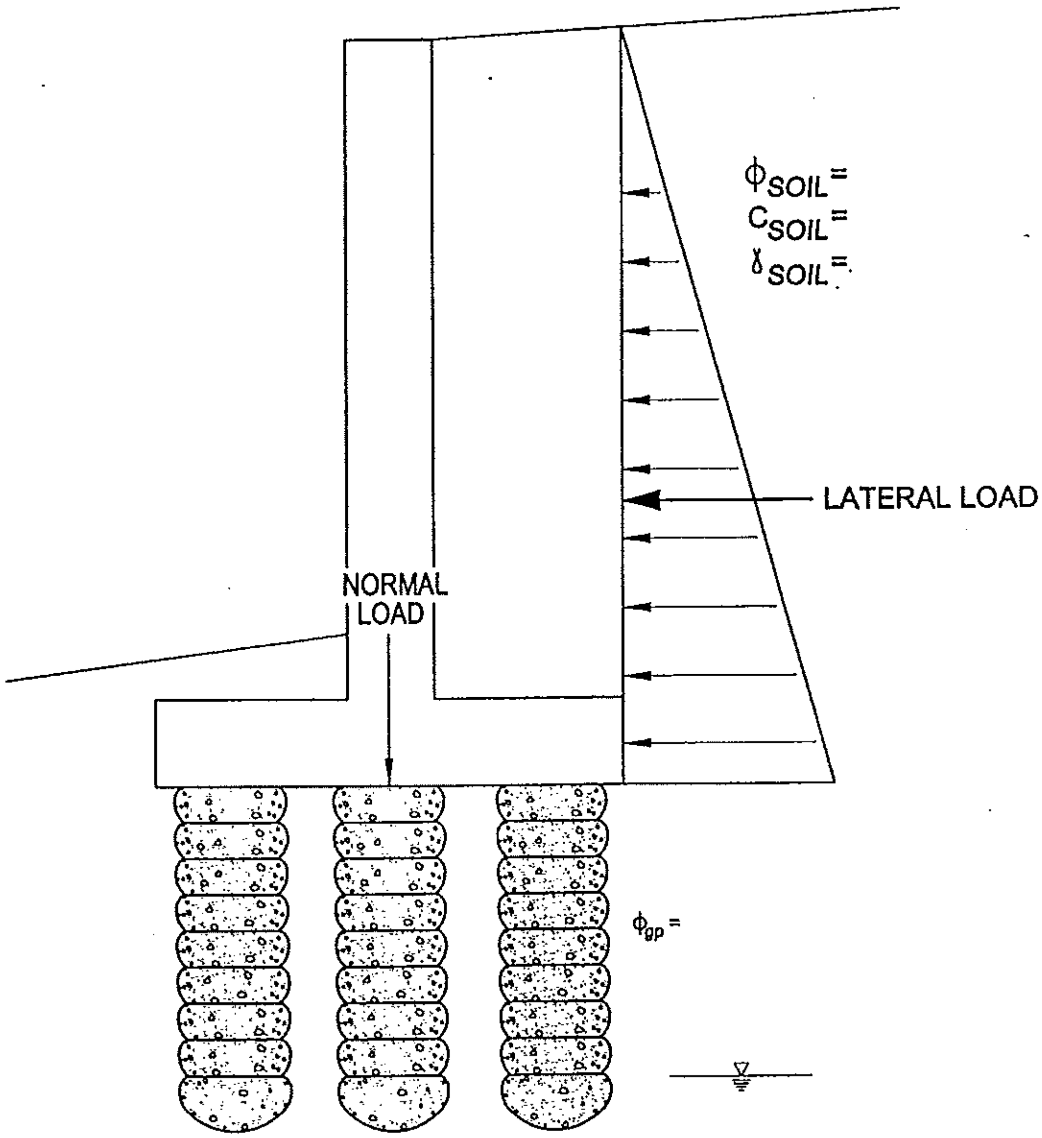


Figure 3.1.1b. Information Required for Sliding Resistance

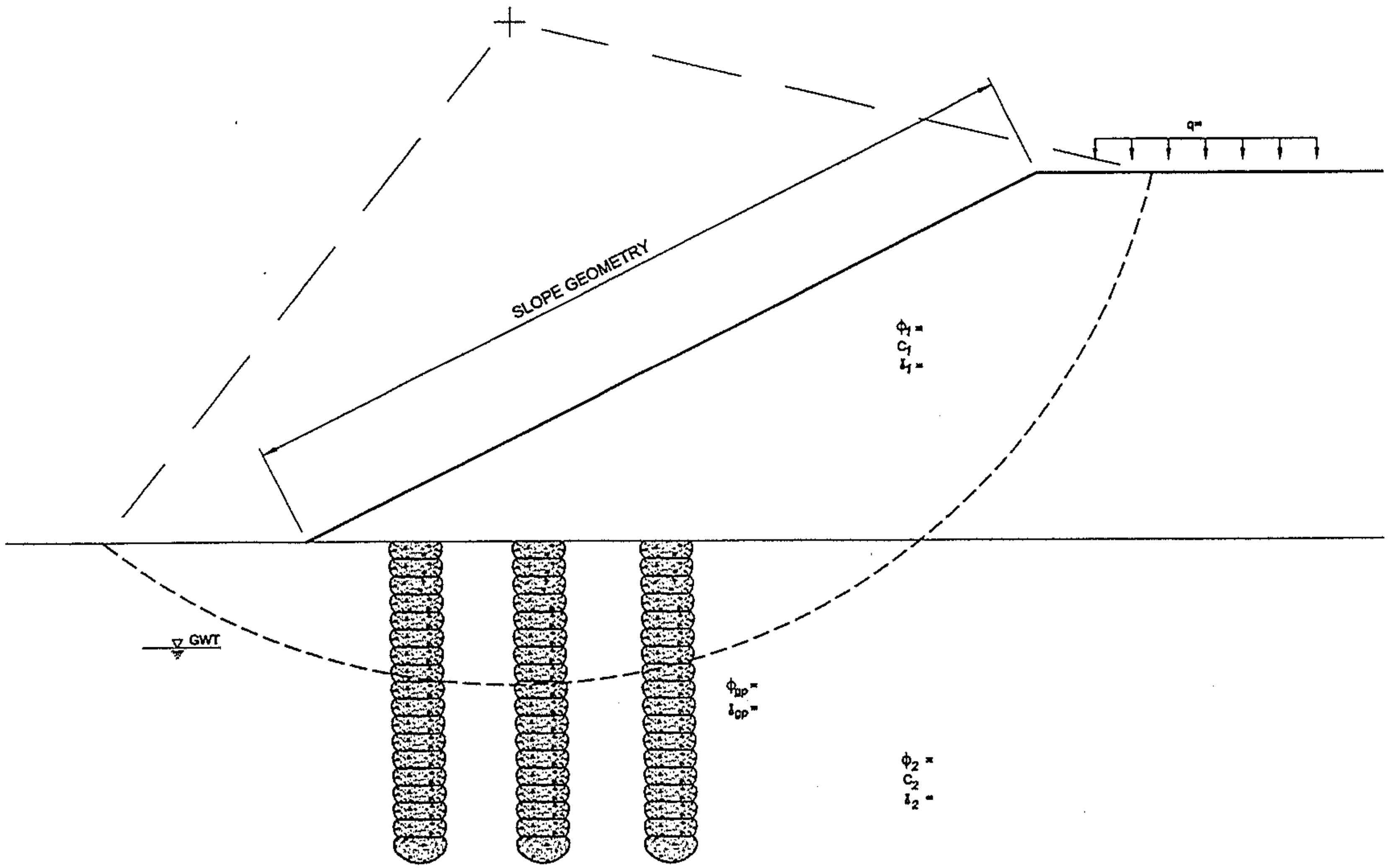


Figure 3.1.1c. Information Required for Slope Stabilization

Strength History

In addition to knowing the properties of the matrix soils, it is also important to have an understanding of the stress history of the soil. If the soils have been prestressed (i.e., are overconsolidated), this will reduce the potential for settlement. Alternatively, if the soils are still undergoing consolidation (i.e., are underconsolidated), which can be the case for waste fills and relatively young fills, then this needs to be considered in the analyses being performed.

Groundwater

When performing a site investigation for a potential Geopier foundation project, knowledge of the groundwater table is important in order to determine if casing or a special gradation of aggregate may be required. It is also important to know if there is the potential for significant groundwater fluctuations that may add load to the structure in the future.

3.2 Geopier Intermediate Foundation Properties

The properties of a Geopier intermediate foundation are a function of the following conditions:

- **Characteristics of the Matrix Soil** – As the strength of the matrix soil increases, the capacity and modulus of the Geopier element that can be constructed also increases.
- **Gradation of the Aggregate** – While open-graded aggregate is most effective for construction of the bottom bulb, and clean aggregate is used for portions of the element built below the water table, the better graded the aggregate, the higher the Geopier element modulus and capacity that can be obtained. Highway base course stone is utilized wherever possible within Geopier elements.
- **Size and Power of the Tamping Equipment** – In general, the greater the force per unit area, the higher the capacity of the Geopier element up to a certain limit, after which the capacity will either remain essentially the same, or may decrease if excessive energy is applied. The unit force is a combination of tamper diameter and the impact of the modified hydraulic hammer.

3.2.1 Geopier Intermediate Foundation Capacity

The capacity of a Geopier element is expressed as *the load carrying capacity of the Geopier element and its representative footing,* mat, or slab segment supported by the matrix soil. For example, for a 3-foot 6 inch square footing supported by one 30-inch diameter Geopier element, the Geopier foundation capacity would be the combination of support provided by the 4.9 square feet area of the Geopier element and the 7.3 square feet area of the matrix soil beneath the footing. For a 10:1 Geopier element to matrix soil stiffness ratio, the support provided for a footing with a 6,000 psf allowable bearing pressure would consist of 13,000 psf from the Geopier element and 1,300 psf from the matrix soil, with the 40% footprint coverage of Geopier element-to-footing area provided in this example.

While the preliminary Geopier element/matrix soil capacity is estimated using Table 4.2, the actual capacity of the Geopier element and its representative footing segment is obtained and confirmed for a project by performing a field modulus load test to determine a conservative measure of the Geopier element stiffness modulus and re-calculating estimated total settlements. Typical modulus load test data are provided on Figure 5.2.3. The modulus load test will confirm the design stress on the Geopier element which will produce tolerable Upper Zone deflections. From these data, the capacity for the Geopier element and matrix soil can be determined, and total settlement estimates can be verified.

3.2.2 Stiffness Modulus of a Geopier Element

The stiffness modulus of a Geopier element, k_{gp}, is defined as the ratio of applied design stress divided by the corresponding displacement (Figure 5.2.3). The modulus represents a conservative estimate of the stiffness of the Geopier element under the design stress. Utilizing this value and the finite grid method of analysis (Bowles, 1980), settlement within the Influence Zone of the Geopier element, or "Upper Zone," is calculated.

The modulus of the Geopier element as obtained from the modulus load test is conservative for the following reasons:

- **Contributions from Matrix Soil Deformation**– By definition, the modulus of a Geopier element is the stiffness of the Geopier element alone. However, because the Geopier element cannot be isolated from the matrix soil during a modulus load test, the load versus deformation behavior measured will be a combination of Geopier load deformation (stiffness) characteristics and matrix soil load deformation characteristics which occurred at that stress intensity. As the stress further increases, deformation of the soil below the bottom of the Geopier element and within the Lower Zone may begin to increase substantially, producing another error on the side of over-conservative Geopier element deformation readings. The true modulus of a Geopier element will be higher than that measured by the modulus load test, since the load test does account for these the matrix soil deformations.
- **Aging of a Geopier Element and Matrix Soil** – The capacity and modulus of a Geopier element in all cases, except the rare case of clays which exhibit negative pore water pressure, is at a minimum immediately after the Geopier element is built. This is because of the build-up of pore pressure (see Section 3.4.2) during installation, as well as isotropic soil characteristics. As pore pressure dissipates, the Geopier element becomes stronger, and the capacity and modulus increase. The modulus load test is typically performed 4 days after construction to help minimize this phenomenon. The effects of soil permeability and moisture content will influence the actual increase in modulus with time, particularly in fine- grained soils.

3.2.3 Shear Strength of a Geopier Element

Since Geopier foundation systems are built using well-graded crushed stone or recycled concrete,compacted to a very high relative density by impact, ramming action, they will naturally have a high shear strength.

In-situ direct shear testing has been performed on Geopier elements built with an open-graded AASHTO No. 57 stone and with a well-graded AASHTO No. 21A base course stone. The No.57 stone is typically used in Geopier element construction for bottom stabilization and construction below the water table, while highway base course stone is normally used for the shaft length above the bottom bulb and above the water table. The results of these tests are provided in Figure 3.2.3. The effective friction angle for the No. 57 stone was 48.8 degrees. The effective friction angle for the No. 21A stone was 52.2 degrees. These values are consistent with those found in the literature (Lambe, 1969) for highly densified, well-graded stone.

3.3 Geopier Element/Matrix Soil Interaction

3.3.1 Lateral Stress Build-Up

The mechanics of constructing a Geopier foundation system consist of:

- Creating a cavity;
- Building a bottom bulb using impact ramming action, vertically prestressing and prestraining the soils below the bottom bulb;
- Building up an undulating shaft by densifying short lifts of well-graded stone, while ramming this stone laterally against the confined sides of the hole, laterally prestressing and prestraining the matrix soils while building up horizontal soil stresses;
- Building a very stiff aggregate element.

This process is simple and quite easily visualized. Ramming well-graded highway base course stone against the side walls within a drilled hole with a high-energy impact tamper efficiently "pushes" the stone against the soil. Meanwhile, the soil "pushes" back, building up passive soil pressures against the stone. Rather than having a retaining wall pushing into the soil, the classic example of the development of passive soil pressures, we have high-frequency, impact energy in a tamper pushing aggregate against portions of a soil wall. The passive pressures generated constitute a permanent stress build-up within the matrix soils adjacent to the Geopier element. These built-up stresses are primarily lateral or horizontal, since they oppose the generally lateral or horizontal stresses pushing against the soil walls during construction of the Geopier element. There is a significant lateral stress build-up during construction of a Geopier element for the following reasons:

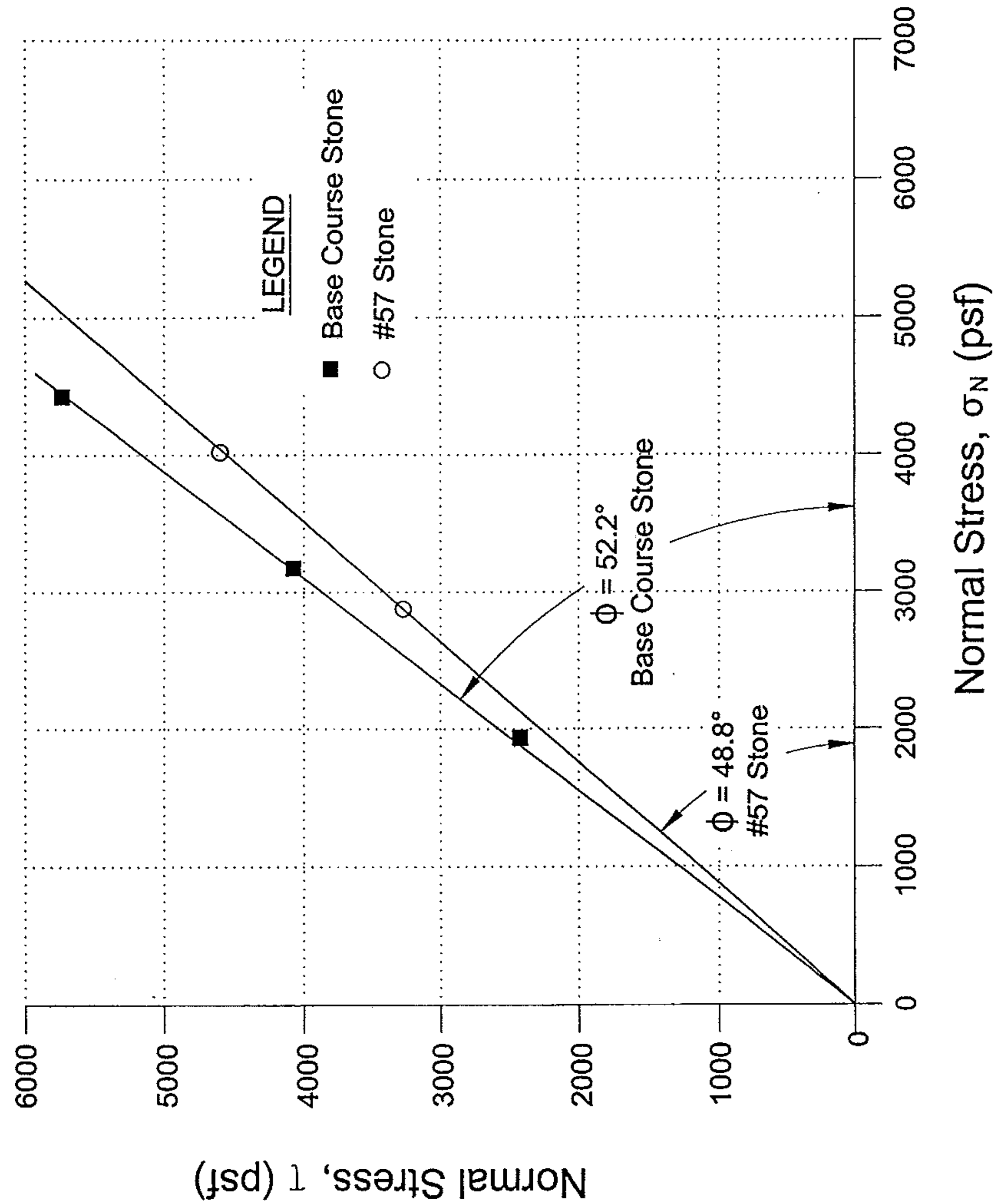


Figure 3.2.3 Shear Strength Envelopes for Geopier Elements

1) The predictable and intuitive mechanics described above.

2) Measured in-situ lateral stresses of matrix soils adjacent to Geopier elements taken using In-situ Stepped Blade Tests. Tests within several different sites indicate that horizontal stresses within the soil after pier installation approach the soils' passive pressure limits and extend laterally from the pier perimeter for a distance approximately equal to two diameters of the Geopier element (Handy, 1997, and more recent in-situ testing).

3) Uplift pullout tests conducted on selected Geopier elements at a number of sites have exceeded theoretical pullout capacities by as much as 400% to 500%, if the build-up of lateral stresses is not considered. Pullout forces on Geopier elements in sands, silts, and clays have significantly exceeded their theoretical capacities, if passive soil pressure is not considered. These pullout load test results can be accounted for only by factoring in the lateral stress build-up approaching the passive soil pressure limits, k_p.

4) Finite element analyses show the lateral stress build-up phenomena during the construction process for a Geopier element.

The mechanisms involved in constructing a Geopier foundation system are different from those involved in high displacement driven pile construction. The aspect of removing a volume of compressible soil material prior to causing lateral soil stress impact, rather than forcing the same volume of soil into the matrix soil extending outward from the pier diameter, provides the advantage of avoiding the creation of extreme excess pore pressure, rupturing the soil structure, and causing general shear failure within the volume of soil physically displaced.

Lateral stress build-up in matrix soils enhances the engineering properties of the matrix soils and contributes to the ability of the Geopier foundation/matrix soil system to control settlements, as well as to provide effective uplift resistances. The lateral stress in the matrix soil is essentially normal stress perpendicular to the shear resistance provided by Geopier elements to either resist downward forces to control settlement, or resist uplift forces to control uplift.

3.3.2 Pore Water Pressure Build-Up

When matrix soils have a low permeability, the shearing resistance of the soil can be temporarily reduced by the creation of excess pore water pressure from the lateral "pushing" of the tamper during construction of a Geopier element. Consolidation will proceed with the passage of time, resulting in an increase in the shear strength of the matrix soil. When consolidation is near completion, the shear strength of the soil will likely exceed its original strength as a result of the increased soil densification applied by installation of the Geopier element.

In addition, during construction of the element, not only is the shear strength of the soil affected, but the lateral stress build-up is also affected by creation of excess pore water pressure. Total pressure is equal to effective pressure plus pore water pressure. The effective pressure or effective stress provides normal stress to the shear plane created at or outside of the Geopier/matrix soil interface. The greater the normal stress, the greater the shearing resistance to displacement that is provided. As the pore water pressure dissipates in time as a result of consolidation, the effective stress increases, and the shearing resistance provided by the Geopier foundation system increases proportionally.

Conditions causing the greatest creation of excess pore water pressure are in saturated, low permeability soils. Therefore, for Geopier elements at and below the groundwater table in clays, clayey silts and low permeability silts, this phenomenon is particularly pronounced, and strength gains in time will be substantial. Conversely, for coarse-grained matrix soils with few fines, this phenomenon may be unimportant. An in-between situation occurs with partially saturated soils containing appreciable fines above the groundwater table. In the latter cases, low to moderate influence may occur, and strength gains from consolidation should be measurable, but may not be substantial.

3.3.3 Strength Gain with Time

Questions have been raised concerning the loss of lateral stress following construction of a Geopier element and the effect of this loss on skin friction over a period of time.

It is true that in time, volumetric creep or secondary consolidation can be expected to reduce the intensity of the built-up lateral stress and its effect on skin friction. It is also true that after disturbance, both fine-grained soils and coarse-grained soils gain strength with aging. These two time-related effects on the Geopier element, one acting to decrease, and the other acting to increase the support capacity of Geopier elements, tend to cancel each other out with time.

To date, there has been little published data to quantify these time-dependent relationships. One study, which included the use of In-situ Stepped Blade and pressuremeter tests performed in the relatively stiff and expansive Houston clay, indicated a 50% decrease in lateral stress during a time period exceeding 100,000 years. Dr. Richard L. Handy has recently performed a study to determine if one of the two phenomena govern over the lifetime of a supported structure (Handy, 1997). The conclusions from this study are summarized below:

1) The Ko stress ratio of any soil, but particularly of clays, should move towards 1.0 over a long period of time. According to Mitchell (1993, p. 325): "It is reasonable to expect that slow changes in lateral pressure will occur in any material that is susceptible to creep and stress relaxation behavior."

2) Horizontal and vertical stresses should logically tend to equalize with time, but the time required is on a logarithmic scale and is measured in tens of thousands of years.

3) Tests made in-situ in a buried stiff expansive clay in Houston indicate that Ko reduced 70% in a soil stratum that was too old for radiocarbon dating (no more than 100,000 years old) (Handy, et al., 1982).

4) At the other extreme, Mitchell (1993) indicates that in soft, uncompacted San Francisco Bay mud which is very susceptible to creep, Ko can change from 0.5 to 0.6 in one week, to 0.7 in 10 weeks, 0.8 in 100 weeks (2 years), etc. This would give a 70% change in less that 20 years instead of 100,000 years for a compact clay. The intense lateral compaction of soil during Geopier installation, therefore, should increase the time required for stress relaxation.

5) Acting in opposition to the stress decrease is a simultaneous, relatively-fast, time-related increase in soil strength. This is shown in pile driving and re-driving where it is called a "set-up factor," and often exceeds 200% in a week (Spangler and Handy, 1982). Another indication of strengthening with time is an upward shift of the measured "preconsolidation" pressure in older soils. The relation between strength and disturbance is called sensitivity, and is 200-400% for "medium sensitive" clay. The strength gain, therefore, should more than offset the reduction in lateral stress with time. The changes are slower in sands than in clays, but sands also can double in strength on aging.

6) Soil has "memory," particularly of the stress history it has undergone. Consolidation tests and in-situ tests, such as with the Menard Pressuremeter, Dilatometer, and Stepped Blade clearly show past maximum effective vertical and horizontal stresses felt and retained by soil deposits. In the vertical direction, these are defined by preconsolidation pressures and overconsolidation ratios (OCR) that are recognized as highly beneficial for increasing foundation bearing capacities and minimizing settlement. Even though these descriptors are only infrequently applied for horizontal stresses, probably because of the difficulty of their measurement, the analogous soil behavior unquestionably exists horizontally as well as vertically. Geopier construction emphasizes soil improvement through the application of both vertical and horizontal stresses with an expanding matrix of crushed stone that is rammed in in layers. Geopier element construction therefore changes the soil fabric and structure in the *primary reinforced zone* by ramming crushed stone into the soil, densifying the soil and causing high increases in horizontal soil stresses. The soil stress history is also changed in the *secondary reinforced zone* by the increase of horizontal stresses and vertical stresses from the ramming energy during construction. The primary reinforced zone is considered to extend 6 inches laterally from the edges of the excavated Geopier cavity. The secondary reinforced zone has been found to extend more than 5 feet laterally from the edges of the excavated Geopier cavity in weak clay soils. These changes in soil fabric, structure, and stress history are imprinted on the soil indefinitely.

<u>Conclusion</u>: The beneficial high Ko resulting from the Geopier densification process will decay with the logarithm of time, but the reduction is more than offset by the increase in soil strength on aging. Part of this is a rapid strength gain that occurs through dissipation of excess pore water pressures. Some excess pore water pressure buildup is inevitable during the construction of a Geopier element in soils with a measurable amount of fines and a moderately low to low permeability. As the pore pressure dissipates near the perimeter of a Geopier element, effective stresses increase. This results in an increase in effective lateral stress with time. *Normally, the weakest time in the life of a Geopier element is when it is first constructed.* The Geopier element gains strength and stiffness in time. One anomaly is the relatively rare situation of negative pore water pressure clays.

4.0 GEOPIER INTERMEDIATE FOUNDATION DESIGN METHODOLOGY

4.1 Introduction

While the primary use and focus of Geopier foundation design to date has been for settlement control of building foundations, Geopier foundations are being used for a number of other applications including:

- **Uplift Control** – Geopier uplift anchors are used for footings and mats subject to uplift forces. Projects completed include airplane hangars, mid-rise buildings, retaining walls and towers.
- **Increasing Allowable Bearing Capacities** – Allowable bearing capacities of building foundation footings are increased significantly, i.e., typically 2 to 6 times using Geopier-supported footings
- **Soil Reinforcement** – High shear strength of Geopier elements significantly increases the slope stability safety factor for walls and embankments reinforced by Geopier elements.
- **Lateral Load Resistance** – The high stress concentration on Geopier elements and the high coefficient of friction between Geopier elements, and footings placed directly on Geopier elements, result in Geopier foundation systems providing significant resistance to sliding for footings subject to static and dynamic lateral loads.
- **Slab Support** –Geopier foundations are used to control settlements for floor slabs on weak or variable soils as an alternate to overexcavation or deep foundation support.

Design of each of these applications takes advantage of one or more of the key characteristics of Geopier reinforced soils:

1) **High Stiffness Modulus of the Geopier Element** – The high Geopier element modulus results not only in providing settlement control, it also results in concentrating normal stresses on the Geopier elements. This stress concentration results in higher effective shear strength for sliding resistance, slope stability, and uplift resistance.

2) **High Shear Strength of the Geopier Element** - Because Geopier elements have an effective friction angle in excess of 50 degrees, they are very efficient for increasing the sliding resistance of Geopier-supported footings and increasing the shear strength of Geopier-reinforced matrix soils.

3) **Lateral Stress Build up in the Matrix Soils**- Because the lateral stress increase within matrix soils adjacent to Geopier elements equals or approaches the passive pressure limit of the soil, the uplift resistance of a Geopier element is significantly higher than a caisson, pile, or other embedded uplift anchor of similar size, per foot of depth. Lateral stress build up increases perimeter shear resistance, which contributes to both settlement and uplift control.

Design methodologies that incorporate these unique characteristics of the Geopier foundation system for the key Geopier applications are discussed in the following sections.

4.2 Settlement Control – Isolated Footings

The design of Geopier foundation systems requires knowledge of the structural loading, site grades and subsurface conditions for a project. The typical information needed for a column footing and a strip footing are shown on Figures 3.1.1a and 3.1.1b.

Once the structural and geotechnical information are obtained, estimating settlement consists of calculating the settlement for two zones beneath the footing as shown on Figure 3.1.a. Settlement in the Upper Zone is controlled by the composite stiffness modulus of the Geopier elements and enhanced matrix soils, and the calculated stress applied to the Geopier element and to the matrix soils. This settlement contribution is determined based on the Finite Grid method (Bowles, 1988). Settlement of the Lower Zone is controlled by the consolidation and compression characteristics of the soil layers below the Upper Zone and the vertical footing stress intensity that reaches this zone. The total estimated settlement of the Geopier-supported footing is the summation of the Upper Zone and Lower Zone settlement contributions.

The specifics of settlement calculations for both Upper and Lower Zone are discussed below.

4.2.1 Upper Zone Settlement – The Upper Zone is defined as the zone equal in depth to the excavation depth of the cavity for the Geopier element below the bottom of footing, plus a depth increment equal to one diameter of the Geopier element. The basis for the addition of one diameter to the Upper Zone depth is to account for the additional depth of influence associated with the bulb created at the bottom of the Geopier element and the vertical prestressing and prestraining associated with construction of the bottom bulb. In soft soils, as shown on Figure 2.1.3, this bulb's thickness itself can be substantially greater than one diameter. In stiff soils, the bulb created can be significantly less than one diameter in thickness, but high intensity vertical prestressing and prestraining effects extend beyond one diameter. In both cases the depth of the Upper Zone as defined is considered to be a conservative estimate because of the depth of influence of the tamper energy on soil below the Geopier element excavation depth.

The settlement contribution within the Upper Zone below the footing is a function of the modulus of the Geopier element, the concentrated stress on the Geopier element, and to a lesser extent, a function of the matrix soil modulus and stress on the matrix soil. As shown on Figure 2.1.4, the Geopier element acts as a stress sink. Because of this, the actual stress on the Geopier element will be significantly higher than the stress on the matrix soil. This is shown by assuming that the footing is perfectly rigid and subsoils within the Geopier-reinforced Upper Zones behave elastically. With the assumption of perfect rigidity in the footing member, equal footing deflections must occur beneath the footing above the very stiff Geopier elements and above the less stiff matrix soils. For static equilibrium to occur, stresses within the footing must redistribute and concentrate on the Geopier foundations according to the following equation:

$q_{gp} = R_s \times q_m$, and **Eq. 8**

$q = q_m \times R_s \times R_a + q_m (1 - R_a)$ **Eq. 6**

Substituting Eq. 8 into Eq. 6 results in :

$q_{gp} = q \; R_s/(R_a R_s - R_a + 1)$ **Eq. 9**

Where:

q = average footing design bearing pressure
q_{gp} = bearing stress on the Geopier element
q_m = bearing stress on matrix soil
R_s = modulus (stiffness) ratio of the modulus of a Geopier element (k_{gp}) divided by the matrix soil modulus (k_m)
R_a = area ratio = Geopier element area (A_{gp}) divided by the total footing area (A)

Once the stresses on the Geopier element(s) and the matrix soil are calculated, the settlement of the Upper Zone (S_{uz}) can be calculated as follows:

$S_{uz} = q_{gp}/k_{gp} = q_m / k_m$ **Eq. 10**

For example:

Given:

Soil Conditions: sandy silt matrix soil with an allowable bearing pressure of 3000 psf and an average N value of 9.

The settlement contribution within the Upper Zone is determined as follows:

q = 7000 psf from Table 4.2
k_{gp} = 260 pci from Table 4.2

k_m = 3000psf / 144 sq. in / sf / 1 in = 20.8 pci

Rewriting Equation 5, R_s = 260 pci / 20.8 pci = 12.5

R_a = 0.33 (assume minimum Geopier element
coverage of 33% for an isolated column footing. Use exact area ratios when footing sizes have been determined or are being assumed for design.)

TABLE 4.2 - Preliminary Values for Geopier^{TM} Foundation Design*

SPT = N Blows Per Foot All Soils	UCS, psf Fine-Grained Soils	Sands & Sandy Silts			Silts & Clays			Peat		
		Allowable Composite Footing Bearing Pressure, psf(1)	Geopier^{TM} Element & Footing Segment Capacity, kips(2)	Geopier^{TM} Element Stiffness Modulus, pci(3)	Allowable Composite Footing Bearing Pressure, psf(1)	Geopier^{TM} Element & Footing Segment Capacity, kips(2)	Geopier^{TM} Element Stiffness Modulus, pci(3)	Allowable Composite Footing Bearing Pressure, psf(1)	Geopier^{TM} Element & Footing Segment Capacity, kips(2)	Geopier^{TM} Element Stiffness Modulus, pci(3)
1-3	200-1000	5000	65	165	4500	50	125	3500	30	75
4-6	1001-2300	6000	90	225	5000	70	175	4000	45	110
7-9	2301-3500	7000	105	260	6000	85	210	5000	55	125
10-12	3501-4600	8000	115	285	7000	100	250	N/A	N/A	N/A
13-16	4601-6000	8500	125	310	7000	105	260	N/A	N/A	N/A
17-25	6001-8000	9000	130	325	7500	110	275	N/A	N/A	N/A
Over 25	Over 8000	10,000	145	360	8000	120	300	N/A	N/A	N/A

Notes: 1. Minimum 30% Geopier^{TM} element area to footing area footprint coverage
Minimum footing sizes: one Geopier^{TM} element, 3' x 3'
two Geopier^{TM} elements, 3' x 6'
three Geopier^{TM} elements, 6'-0" x 6'-0"

2. For 18" Geopier^{TM} elements, multiply by 0.45
For 24" Geopier^{TM} elements, multiply by 0.7
For 36" Geopier^{TM} elements, multiply by 1.3

3. Geopier^{TM} element modulus to be confirmed by full-scale load test
For underconsolidated clays, multiply modulus values by 0.67
For overconsolidated clays, multiply modulus values by 1.10

*For isolated spread footings supported by 30" diameter Geopier elements

Using Equation 6 and solving for q_m and q_{gp}:

$q = 7000$ psf $= q_m (12.5)(0.33) + q_m (1\text{-}0.33)$
$q_m = 1460$ psf $= 10.1$ psi

$q_{gp} = R_s\, q_m = 12.5 \times 1460$ psf $= 18{,}250$ psf $= 126.7$ psi

Using Equation 10, the Upper Zone settlement S_{uz} is calculated as follows:

$S_{uz} = q_{gp} / k_{gp} = q_m / k_m =$ 126.7 psi/260 pci = 10.1 psi/20.8 pci = 0.49 inches

Note that the Upper Zone settlement is independent of the length of the Geopier element since the modulus is in the form of a subgrade modulus. This assumption is considered to be reasonable for Geopier depths up to 5 times the Geopier diameter and may become less theoretically accurate with deeper Geopier elements. It has been found in practice that use of this model for Geopier elements up to 15 feet in length have provided reasonable and conservative settlement predictions.

One reason for the conservative predictions is the fact that the actual modulus values for the Geopier elements are higher than the values obtained from field modulus load tests. The modulus values as measured in the field are influenced by Lower Zone strains that occur in the soil strata below the Geopier element. This is a major factor in limiting maximum loads for modulus load tests to produce 150% of the design stress of the Geopier element, since the influence of the soil compressibility characteristics below the Geopier element becomes more of a factor, adding to over-conservative errors in measuring Geopier element stiffness modulus values.

Conservative factors, or "safety factor elements," in this methodology which result in less settlement than predicted within the Upper Zone include:

1) The true modulus of the Geopier element is greater than that measured in the modulus load test because of Lower Zone influences discussed above and also because the Geopier foundation cannot be isolated from the adjacent matrix soil. As a result, the modulus measured is partially the modulus of the Geopier element and to a lesser extent, partially the matrix soil modulus.

2) The use of allowable bearing pressures for small footings as the basis to calculate matrix soil modulus values for design stress calculations contains an inherent, built-in safety factor of 1.5 to 2.0.

3) The confining effect of the footing itself, which provides additional lateral stresses created by the footing's vertical stress on the matrix soil, is not taken into account. The additional lateral stress further stiffens the matrix soils underlying the loaded footing.

4) The increased stiffness of the improved matrix soil is not considered.

5) The increase in Geopier foundation element strength and stiffness in time, whereas the modulus load test is typically performed 3 or 4 days after it is installed.

4.2.2 Lower Zone Settlement – The Lower Zone is defined as the vertical extent of soils below the Upper Zone to a depth of influence based on the size and shape of the loaded area of the footing being supported. The Lower Zone thickness becomes the total depth of significant stress influence for a footing minus the Upper Zone depth. The total depth of significant stress influence can be calculated as follows:

For isolated footings:

Square Footings	2 x B where B is the footing width
Rectangular Footings (with length-to-width ratios equal or less than 4.0)	2 x B' where B' is the square root of the footing width times the footing length

For Strip or Continuous Footings = 4 x B where B is the footing width or the diameter of the Geopier element, whichever is greater.

Settlement contribution within the soil in the Lower Zone is a function of the thickness of the Lower Zone, the stress intensity from the footing stresses on the Lower Zone, and the compressibility and consolidation characteristics of the Lower Zone soil.

4.2.2.1 Lower Zone Settlement Analysis Methods

Methods used to estimate settlement contributions within the Lower Zone depend on the subsoils present, the data available, experience of the Geopier design engineer, and local geotechnical engineering experience. Methods that are being used and have been successfully used since 1991 include: Consolidation theory, Schmertmann layer strain method, and modified elastic theory methods. Data used to provide information for analyses include: consolidation data, standard penetration test data, unconfined compression test data, static cone penetrometer data, in-situ Stepped Blade data, and pressuremeter data.

A number of factors contribute to an effective spreading out of the load (and therefore, reduction of vertical stress intensity) at the interface separating the Upper Zone and the Lower Zone. These include:

1) A substantially stiffer Upper Zone overlying a more compressible Lower Zone. The stiffness of the composite Upper Zone includes contributions from the inclusion of the very stiff and high modulus Geopier elements, and additional contributions from the substantial lateral stress build-up which occurs within the matrix soils during Geopier foundation installation operations.

2) Significant vertical prestressing and prestraining within and below the bottom bulbs of the Geopier elements, which act as a "preload" to reduce the magnitude of new stresses felt by the Lower Zone soils application of structure load.

3) Stress distribution of the concentrated stresses within Geopier elements have been recently measured by instrumented load cells. Stress dissipations within the Geopier elements have been determined to be considerably greater (faster) than shown by theoretical stress distributions from footing bottoms bearing on unreinforced soils, regardless of whether the distributions are represented by Boussinesq or by Westergaard stress theories.

Stress distribution assumptions, which have been used selectively in estimating Lower Zone settlement contributions and to partially compensate for the over-conservative and erroneous assumption that the combined Upper Zone and Lower Zone Geopier reinforcement system represents a homogeneous and isotropic situation, include the following:

- Schmertmann's Layer Strain stress distribution with results multiplied by 0.67;
- Boussinesq stress distribution with results multiplied by 0.80;
- A simple 1.67 V:1.0 H rule, analogous to the Boussinesq 2.0 V:1.0 H rule;
- Westergaard stress distribution with Poisson's ratio of 0.

Often multiple analyses are performed with computer assistance, and comparisons of settlement contributions are made.

It is important to note that the two-layer, Upper Zone, Lower Zone method has been used for settlement estimates since 1991, and during this time no Geopier-supported structure is known to have settled more than estimated. This includes use of all of the methods discussed above to estimate Lower Zone settlement contributions. Based on this relatively extensive feedback, we can conclude that the methods presently being used can be considered reasonable and conservative. It is anticipated that if and when any modifications in Lower Zone settlement estimating methods are made, they will likely be to reduce the over-conservative nature of settlement prediction by better understanding the actual stress distributions and the effects of "preloading" or prestressing on the Geopier-reinforced soil system.

4.2.2.2 Stress within the Lower Zone Soil - A conservative way of determining the stress in the Lower Zone is to use an elastic method for determining soil pressure at various points in the soil stratum assuming that the soils in the Upper and Lower Zone have the same properties. Boussinesq theory assumes the soil is elastic, homogeneous, isotropic and semi-infinite. However, the composite Upper Zone of Geopier-reinforced soil is significantly different from the Lower Zone soil because of the inclusion of very stiff aggregate piers, which act as "stress sinks." The Upper Zone soil is also anisotropic, since the matrix soil and the Geopier reinforcement have high lateral stresses and can be classified as having a relatively high over-consolidation ratio (OCR) as a result of the prestressing from the installation process. The composite Upper Zone will typically have a significantly higher elastic modulus, E_s, than the Lower Zone soils, normally 4 to 10 times stiffer if the Upper and Lower Zones had similar soil engineering properties. A more accurate stress dissipation model is one that can take these factors into account.

One method often used to estimate Lower Zone settlement contributions is the Schmertmann Layer Strain approach. This method is more appropriate in granular soils, but has been selectively used in non-granular soil situations as well. An important aspect of using this method is to initiate the stress distribution from footing bottom, even though the method is used exclusively to estimate Lower Zone contribution only. In order to accomplish this, the first subsoil layer should coincide with the interface between Upper Zone and Lower Zone. Settlement contributions from this first layer (the Upper Zone) are then disregarded, and only the underlying strata settlement contributions are considered in the Lower Zone calculations. Total settlement estimates are determined by adding the Upper Zone settlement contribution calculated from the Finite Grid method to this Lower Zone settlement contribution. One way to simplify this is to interject an artificially high modulus in this Upper Zone layer, say 300 blows per foot, and a gravel soil. Settlement contributions from this zone will then approach zero, and typically less than 1 to 4 hundredths of an inch. Even these small amounts are then subtracted from the resulting settlement estimate. Settlement estimate results are multiplied by 0.67, as is common practice in geotechnical consulting practice in Florida by consultants who use the Schmertmann Layer Strain method.

The advantages of this method are simplicity, use of either Static Cone data or Standard Penetration Test data, and the fact that extensive uses of this method for a period of seven years have proven to be realistic and conservative for over one hundred projects and over 15,000 Geopier elements. The method has proven to be over-conservative or excessively conservative when used for mats, strip mats, and heavily-loaded continuous footings. The apparent reason for this is that the spreading out of the load as a result of both stiffer Upper Layer and the prestressing effect of the pier construction results in faster stress dissipation than is assumed, and significant stresses extend less deep beneath Geopier-supported foundations than beneath non-reinforced, bare foundations.

Another method presently used to approximate the stress in the Lower Zone for **isolated footings** is by using the Westergaard theory. The Westergaard's method assumes the soil mass consists of layered soil strata of finer and coarser materials or non-isotropic soils. Using this theory the anisotropic nature of the soil as defined by its Possion's ratio (ratio of lateral strain to vertical strain) can also be taken into account. Figure 4.2.1 provides typical pressure isobars for isolated and continuous footings for a Poisson's ratio of 0. While using Poisson's ratio of 0 may be considered unconservative for the Lower Zone soil, this is more than balanced by the conservative influences of the stiffness of the Upper Zone layer and the fact that the percentage of stress used is taken for the point directly in the middle of the footing, the effects of Geopier element bottom bulb prestressing, and stress dissipation within Geopier elements.
Another method presently used is Boussinesq stress distribution with Lower Zone

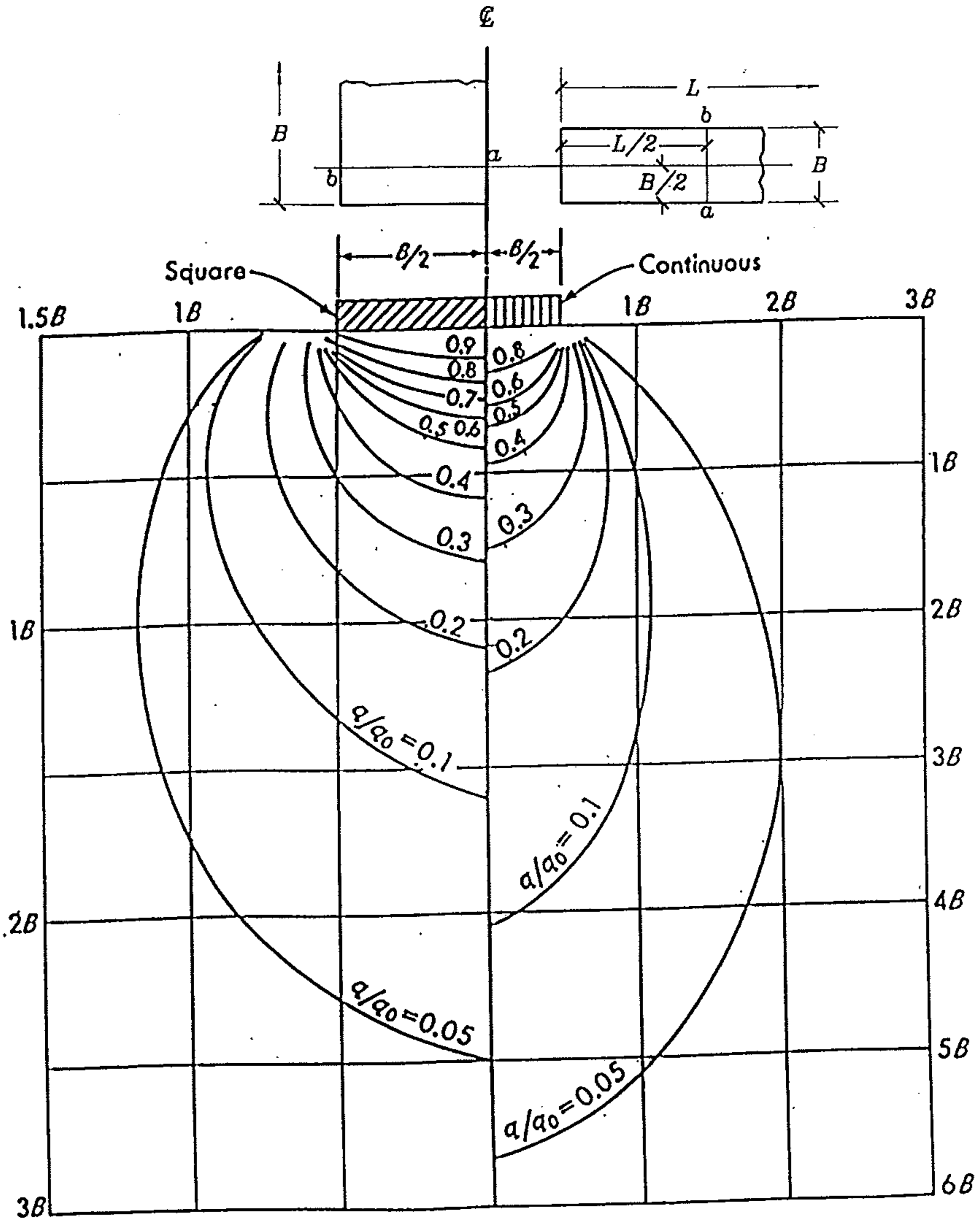


Figure 4.2.1. Westergaard Chart, from "Foundation Analysis & Design", J.E. Bowles, 1977, p. 156

settlement magnitudes multiplied by 0.80 for settlement contribution estimates. A somewhat comparable approach is to use a 1.67 V:1.0 H rule for stress distribution from the footing in order to estimate the effect of spreading out the load.

For relatively heavily loaded **continuous footings or strip mats with a high Geopier element area coverage** (30% or more), an analysis that takes into account the benefits of the stiffer Upper Zone soil is often used. This approach assumes that the footing is supported by a two-layer system in which the upper soil layer is significantly stiffer than the Lower Zone soil and that the layers extend semi-infinite. For a continuous strip footing with closely-spaced, prestressed Geopier elements, and the fact that the stress is taken directly beneath the center of the footing, this may be a reasonably good assumption. Figure 4.2.2 provides a comparison of the stress distribution under a footing assuming a homogeneous mass (Boussinesq) and a two-layer system (Fox, 1948). The figure shows that the stress directly beneath the footing dissipates significantly quicker for the two-layered system. The influence of the depth of the stiff layer also has a significant affect on the stress dissipation. Figure 4.2.3 provides a means of determining the stress at various depths for different layer thicknesses and different ratios of Upper Zone to Lower Zone stiffnesses.

Examples of how two of these methods, Westergaard and two-layered elastic method, are used in settlement estimates for Geopier foundations are provided in the following sections.

4.2.2.3 Compressibility of the Lower Zone Soils

The compressibility of the Lower Zone soils can be estimated using the following data:

- Standard Penetration Resistance Testing (SPT)
- Consolidation Testing
- In-situ Cone Penetration Testing (CPT)
- In-situ Stepped Blade Testing
- In-situ Pressuremeter Testing (PMT)
- Unconfined Compression Testing

The method used to determine the compressibility of the Lower Zone layer for a Geopier foundation analysis is similar to that used for conventional spread footings. The main difference is that there will be considerably less stress on the interface plane separating the Lower Zone than there would be with a bare footing bearing at the same elevation. *If settlement contribution within the Lower Zone is estimated to be greater than desired, the Geopier elements can be lengthened to reduce the thickness of the Lower Zone, thereby reducing the Lower Zone settlement contribution.*

Several common correlation's' to develop the Lower Zone modulus are as follows:

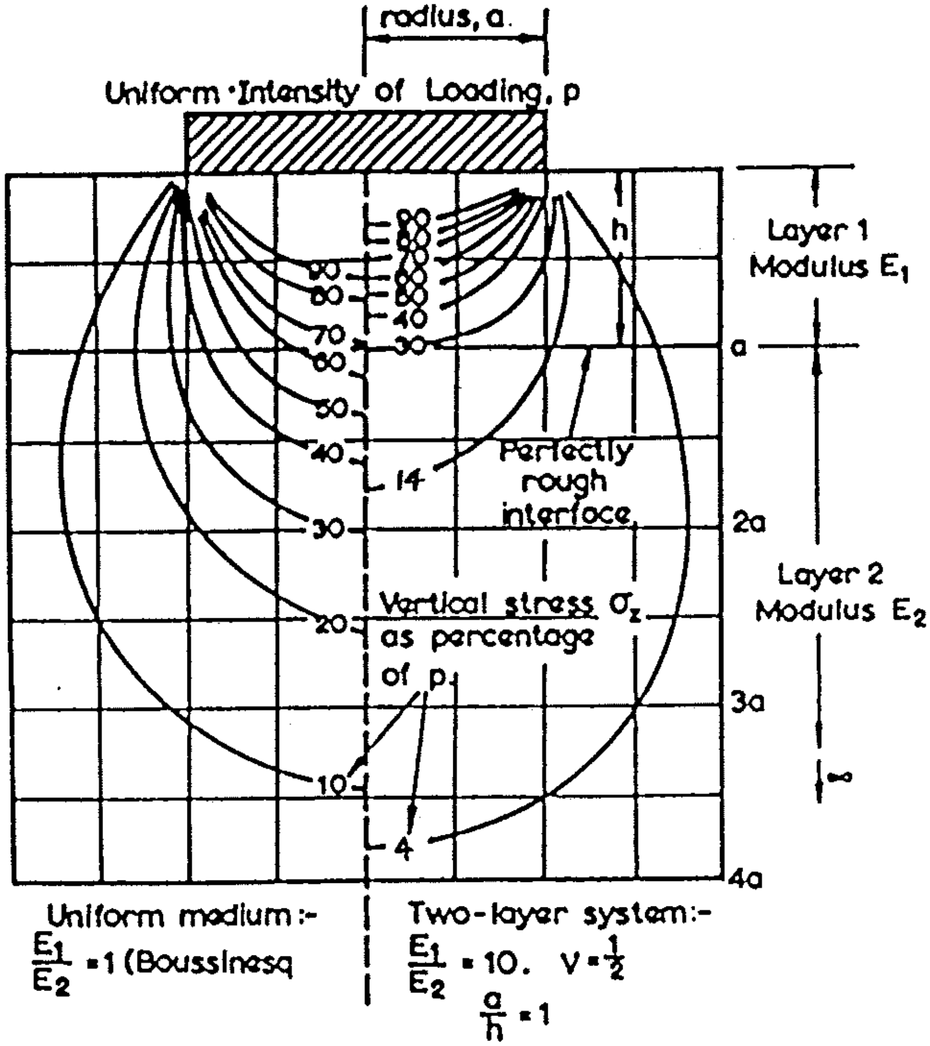


Figure 4.2.2. Stress Distribution Model for Lower Zone (Poulos & Davis)

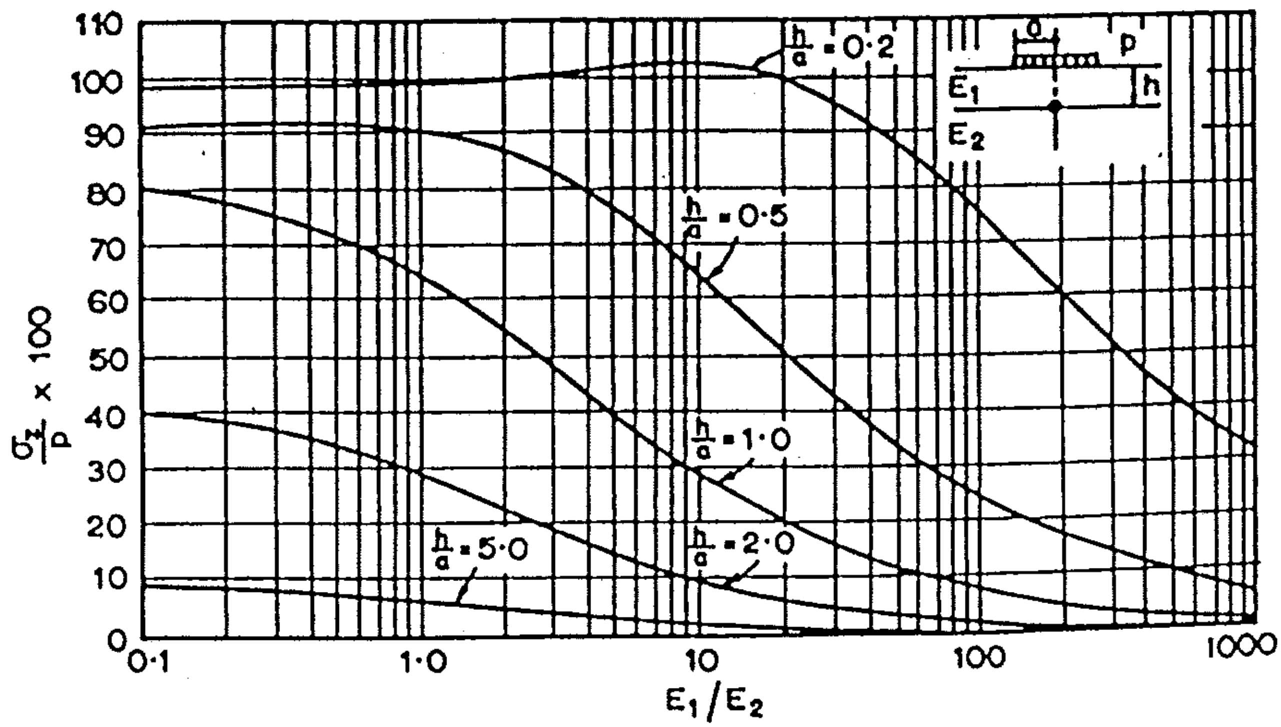


Figure 4.2.3. Stress Distribution Model for Lower Zone (Poulos & Davis)

For residual silts, sandy silts and silty sands in the Piedmont Province (Eastern USA), Martin (1987) correlated results from pressuremeter testing, standard penetration resistance testing and field performance to develop the following equation:

$E_s = \text{Log}^{-1} [1.17627 + 0.70437 \text{ Log } N]/0.6$ (tsf) where N is the standard penetration test blow count,

for example:

$E_S = 152$ ksf @ $N = 5$

$E_S = 252$ ksf @ $N = 10$

Mitchell and Gardner (1975) presented correlations for SPT for sands as follows:

$E_s = 10(N + 15)$ (ksf) for sands

$E_s = 6(N + 5)$ (ksf) for clayey sand

Several sources, as cited in Bowles (1978), correlate cone penetration testing to elastic soil modulus as follows:

$E_s = 3\ q_c$ (units of q_c) for sands

$E_s = (2 \text{ to } 8)\ q_c$ (units of q_c) for clays depending on the cone used and the soil type

Typical value ranges of the static stress strain modulus, E_s, for selected soil types as reported in Bowles (1978) are provided on the following page:

Soil Type	Es (ksf)
Clay	
Very Soft	7 - 58
Soft	28 - 86
Medium	86 – 172
Hard	144 – 432
Sandy	576 - 864
Silt	43 – 432
Glacial till	216 – 3168
Loess	288 – 1152
Sand	
Silty	144 – 432
Loose	216 – 504
Dense	1008 – 1728
Sand and Gravel	
Loose	1008-2880
Dense	2016-4032

NOTE: Values selected will depend on stress history, water content, density, etc.

It is important to remember that knowledge of the stress history of a soil, as stated earlier, is an important aspect of estimating the modulus and compressibility of the Lower Zone soil. For example, for areas where deposits are underconsolidated, such as recent fills and coastal and alluvial deposits, the modulus may be less than the low end of the above tables and equations. On the other hand for overconsolidated deposits, the stress from previous geologic eras may exceed the stress of the new structure. Selection of any design parameters should be based on knowledge of the local geology and soil stress history whenever possible.

4.2.3 Isolated Footings - Square and Rectangular

Using the example started in Section 4.2.1 the design of Geopier foundation support for settlement control of an isolated footing is presented below:

Given:

Soil Conditions: Sandy silt matrix soil with an allowable bearing pressure of 3000 psf and an average N value within the Upper Zone of 9. Lower Zone soils consist of Piedmont Region residual sandy silts with an average N value of 10.

Column Load: 600 kips

From the previous example

q = 7000 psf (or less) from Table 4.2. Use 7000 psf.
k_{gp} = 260 pci (or less) from Table 4.2. Use 260 pci.
k_m = 3000 psf/144 sq. in/sf / 1 in = 20.8 pci
R_s = 260 pci / 20.8 pci = 12.5
R_a = 0.33 (assume Geopier element coverage of 33% for an isolated column footing. Minimum Geopier element coverage accepted is 30.0%)
q_m = 1460 psf
$q_{gp} = R_s\, q_m$ = 12.5 x 1460 psf = 18,250 psf

Therefore **Upper Zone settlement** equals:

$S_{uz} = q_{gp} / k_{gp}$ = **0.49 inches**

Footing Size = 600 kips/ (7000 psf /1000 kips/ lb) = 86 sf

Use 9.25 x 9.25 ft footing size A = 85.6 sf

Influence Depth = 2B = 2 x 9.25 = 18.5 foot depth

Assume 30-inch (2.5 foot) diameter Geopier elements

Assume Initial Geopier element shaft length is 1 B = use 9 ft. shaft length + 2.5 ft = 11.5 ft thickness of Upper Zone

Upper Zone Thickness = 11.5 ft.

Lower Zone Thickness = 18.5 – 11.5 = 7 ft

<u>Using Westergaard Method</u>.

Center of Lower Zone is 18.5 – 7/2 = 15.0 ft = 15.0/9.25 B = 1.6 B
Using Westergaard Charts Figure 4.2.1 Stress at 1.6 B = 10% footing stress

Stress on Lower Zone soil = 600/(9.25 x 9.25) x 0.1 = 0.701 ksf

From Martin's correlation for Piedmont soils the E_s for the Lower Zone soil is

E_s = 126 tsf = 252 ksf

Lower Zone Settlement = 0.701 ksf x 7 ft /(252ksf) = 0.0195 ft

= 0.23 inches

Total Settlement = 0.49 + 0.23 = 0.72 inches

For rectangular footings with length-to-width ratios of 4.0 or less the analysis is the same, and the influence depth is equal to a depth equal to two times the square root of the footing width times the footing length.

Note: Use of Schmertmann Layer Strain approach with a recommended reduction factor of 0.67 results in a Lower Zone settlement contribution of 0.33 inches. Total settlement estimate using the Schmertmann method is 0.49 + 0.33 = 0.82.

4.2.4 Continuous Footings

For continuous footings there are two different types of design cases. The first case is for relatively lightly loaded strip footings (2 to 5 kips per lineal foot) that are typically 2 to 3 feet wide, and the Geopier elements are widely spaced with spacing that may be controlled by either the structural span of the footing or the capacity of the Geopier foundation support. In this case the Geopier element coverage is significantly less than 30% and typically less than 20%. The second case (discussed briefly in 4.2.2.2) is for heavily loaded continuous footings and retaining walls where the Geopier element coverage is normally greater than 30%, and the coverage of the Geopier elements is controlled by the stress on the footing. In this case the two-layered design approach for stress on the Lower Zone may be used since the concentration of Geopier elements approaches a semi-infinite layer in one direction. Settlement Control for each of these cases is discussed below.

4.2.4.1 Lightly Loaded Continuous Footings

For relatively lightly loaded strip footings, the Geopier foundation system layout is normally controlled by capacity of the Geopier element and the structural span of the reinforced concrete footing. Typical Geopier element spacing can vary from about 6 to 20 feet on center depending on the Geopier capacity that can be achieved, wall footing structural design, and the loads. For these conditions the settlement of the Geopier-supported footing in the vicinity of the Geopier element is controlled by the depth of influence of the strip footing, which is considered to be 4 times the footing width (Figure 2.1.3b). For example, a 2.5-foot wide footing will have a significant stress influence to a depth of 10 feet. The stress influence on the Lower Zone will be controlled by this influence depth and the charts for continuous footings in Figure 4.2.1 may be utilized. Using the following soil information a typical design is developed.

Given:

Soil Conditions: Eight feet of a variable fill consisting of firm silty clay exists on site. The choice is to overexcavate the variable fill or use Geopier foundation support. The fill soil has an estimated allowable bearing pressure of 1500 psf and an average N value of 6. Lower Zone soils consist of Piedmont Region residual sandy silts with an average N value of 9.

Strip Footing Loads: 3 kips per lineal foot

q = 5000 psf or less from Table 4.2. Use 5000 psf.
k_{gp} = 175 pci or less from Table 4.2. Use 175 pci.
k_m = 1500 psf /144 sq. in /sf /1 in = 10.4 pci
R_s = 175pci /10.4 pci = 16.8

From Table 4.2 the **capacity for a 30-inch diameter Geopier element and its representative footing segment <u>for an isolated footing</u> is 70 kips**. This capacity should be reduced to 90%, since the matrix soil support for a continuous footing is less efficient than for an isolated footing. Use a capacity of 0.90 x 70 kips = 63 kips. Sixty-three kips/3 kips per lineal foot = 21 feet. Alternatively, a 24-inch diameter Geopier element would be required every 14.7 feet: (63 kips x .7 = 44.1 kips) 44.1/3 = 14.7 feet.

The designer selected a 14-foot spacing using a 2-foot diameter Geopier element. A footing design width of 2.5 feet was also selected.

Using these data, the following are calculated:

It is assumed that the entire load is bearing on the Geopier foundation and the matrix soil within a footing length equal to 3 times the diameter of the Geopier element, or 3 x 2 ft = 6 ft. This is a conservative assumption which recognizes the significant stress concentration on the Geopier foundation and the fact that most of the load was resisted by the Geopier reinforcement and the enhanced matrix soil. Therefore:

R_a = 3.14/(2.5 x 6) = 0.209
q = 3 x 14 feet /(2.5 x 6) = 2.8 ksf
q_{gp} = 2800 psf x 16.8 /(0.209 x 16.8 – 0.209 + 1) = 10,934 psf
q_m = bearing stress on the matrix soil = q_{gp} / R_s = 10,934/16.8 = 651 psf

Therefore, **Upper Zone settlement** equals:

S_{uz} = q_{gp} /k_{gp} = q_m /k_m = 10,934/175 = 651/10.4 = 62.5 cubic inches/sf

= 62.5 cubic inches/sf x 1sf /144 sq. in.

= **0.43 inches**

Influence Depth = 4B = 4 x 2.5 = **10 foot depth**

Assume a 24-inch (2.0-foot) diameter Geopier element

Initial length of the Geopier element is 2B for continuous footings or 6 foot min. = use 6 foot shaft length + 2.0 ft. (pier diameter) = 10.0 ft. effective Upper Zone thickness.

Upper Zone Thickness = 10 ft.

Lower Zone Thickness = 10 - 8 **= 2.0 ft.**
Therefore, no Lower Zone settlement.

Using Westergaard Charts Figure 4.2.1, Stress at 3.6 B = 12% footing stress

Stress on Lower Zone soil = 2.8 ksf x 0.12 = **0.336 ksf**

From Martin's correlation for Piedmont soils the E_s for the Lower Zone soil is

E_s = 98 tsf = 196 ksf

Lower Zone Settlement = 0.336 ksf x 2.0 ft /(196 ksf) = **0.00343 ft**

0.00343 ft X 12 inches/ft = 0.041 inches = **0.04 inches**

Total Settlement = 0.43 + 0.04 = **0.47 inches**

Note: Use of Schmertmann layer strain approach results in a lower settlement contribution of 0.02 inches. Total settlement using the Schmertmann method would then be = 0.43 + 0.02 = **0.45 inches.**

4.2.4.2 Heavily Loaded Continuous Footings

For heavily loaded continuous footings, strip mats and retaining walls where the coverage is close to or greater than 30% and the coverage of Geopier elements is controlled by the stress on the footing, a two-layered analysis method may be used to develop the stress in the Lower Zone. In these cases the Geopier reinforced zone acts as a continuous (in one direction only) stiff soil layer overlying a less stiff layer. Because of this situation, the stress influence on the Lower Zone will be reduced. The following example illustrates this method.

Given:

Soil Conditions: Sandy silt matrix soil with an allowable bearing pressure of 3000 psf and an average N value of 9. Lower Zone soils consist of Piedmont Region residual sandy silts with and average N value of 10.

Wall Loads: 40 kips per lineal foot

q = 7000 psf or less from Table 4.2. Use 7000 psf.
k_{gp} = 260 pci or less from Table 4.2. Use 260 pci.

k_m = 3000 psf /144 sq. in/sf/1 in = 20.8 pci
R_s = 260 pci/20.8 pci = 12.5

Based upon a load of 40 kips per LF and 7000 psf allowable bearing pressure, the minimum footing width is 5.71 feet. A width of 6.0 feet is selected, which results in a footing stress of 6667 psf. A minimum area ratio of 0.33 is used.

Using these data, the following are calculated:

R_a = 0.33
q_{gp} = 6667psf x 12.5/(0.33 x 12.5 – 0.33 + 1) = 17,380 psf
$q_m = q_{gp}/R_s$ = 17,380 psf/12.5 = 1390 psf

Therefore, **Upper Zone settlement** equals:

$S_{uz} = q_{gp} / k_{gp} = q_m / k_m =$ 17,380/260 = 1390/20.8 = 66.8 cubic inches/sf

= 66.8 cubic inches/sf x 1sf /144 sq. in.

= **0.46 inches**

Next the following assumptions are made to calculate Lower Zone stress:

Geopier shaft length. Use 10 feet because of high loads and resulting deep influence zone.
Upper Zone thickness = 10 ft + 2.5 ft = 12.5 feet
Ratio of the Upper and Lower Zone Es = 4.75. Use 5.0.

Therefore:

Influence depth below footing = 4B = 4 x 6 = 24 feet
Lower Zone thickness = 24 – 12.5 = 11.5 feet
Center of Lower Zone = 24 – 11.5/2 = 18.25 = 18.25/6 = 3.04B = 6.1A (radius)

Using Figure 4.2.3:

For h/a = 12.5/6/2 = 4.2, and E1/E2 = 5.0.

Stress on Lower Zone soil = 5% of the footing stress

= 0.05 x 6667 psf = **333 psf**

From Martin's correlation for Piedmont soils the E_s for the Lower Zone soil is

E_s = 126 tsf = 252 ksf

Lower Zone Settlement = 0.333 ksf x 11.5 ft /(252 ksf) = 0.0152 ft

= **0.18 inches**

Total Settlement = 0.46 + 0.18 = **0.64 inches**

4.2.5 Mats and Slabs

For mats and slabs the extent of the loaded area will affect the influence depth and hence the settlement. However, if the loads can be isolated on the Geopier foundations through drop footings, then the stresses can be better controlled and analyzed, under certain conditions, as individual column or continuous footings.

For heavily loaded structures covering a wide area, such as silos and earth embankments, the Geopier soil reinforcement more approximates a semi-infinite layer. The footings supported by Geopier elements can then be analyzed as a heavily-loaded continuous footing. In general, for cases involving silos and embankments, stresses on the Lower Zone will be high, resulting in larger settlement predictions. However, oftentimes total settlement requirements will be less for these types of structures, with an allowable design settlement being greater than one inch. Differential settlement control of ring footings for grain silos has proven very effective with Geopier foundation support of the ring footings.

4.3 Ultimate Bearing Capacity of Geopier Foundations

Settlement criteria, which requires limiting settlements of footings to total settlements of approximately one inch, typically controls design requirements for Geopier foundations. Bearing capacity seldom, if ever, controls, as excessive settlement will occur prior to a bearing capacity failure for virtually any anticipated Geopier-supported foundation situation. The possible exception to this is the use of Geopier elements to support large diameter tanks bearing on soft subsoils, and a few specialized support systems.

The Upper Zone below a footing supported by Geopier elements consists of a composite zone of both Geopier elements and matrix soil, which is significantly stiffer and stronger than the matrix soil alone. Any potential failure surface including a bearing capacity failure surface must pass through or around this strengthened zone. There are three possible failure mechanisms for bearing capacity failures of a footing supported by Geopier elements. First, the critical failure surface could be a general shear failure passing through the **Upper Zone** and into the matrix soil (Figure 4.3a). Since the angle of internal friction in the densified aggregate within a Geopier element is very high (measured values from 48.8 to 52.2 degrees), for most conditions it is unlikely that the failure surface will pass through the strengthened Upper Zone. The exception to this will be for footings with very high bearing pressures, where Geopier elements are overlying very strong Lower Zone soils.

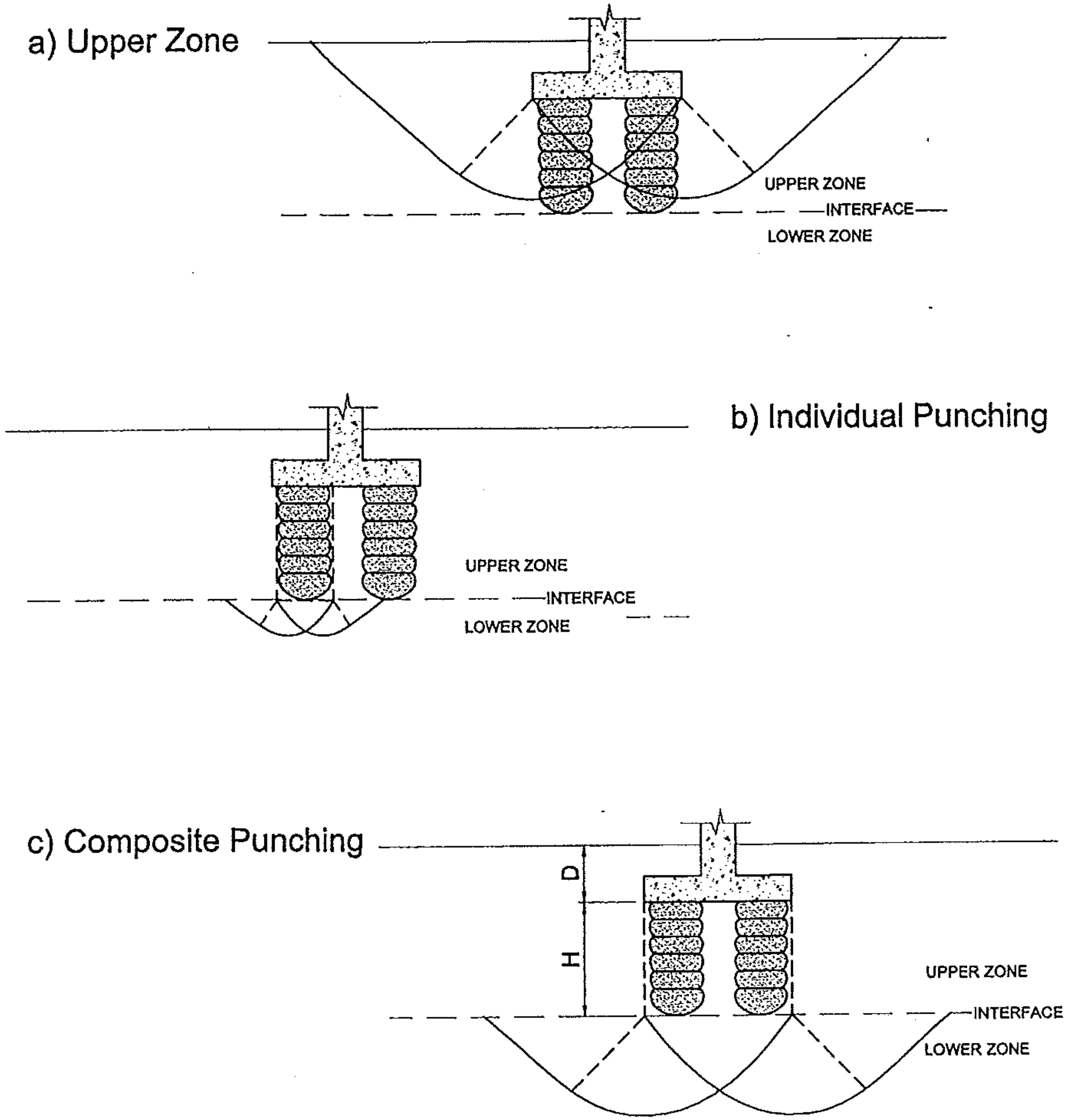


Figure 4.3 Bearing Capacity Failure Modes

A second potential mechanism is for a punching failure to occur within the composite zone itself (Figure 4.3b). This would occur as an **individual punching** failure of a Geopier element punching through adjacent matrix soils at the perimeter of the Geopier elements. A general shear failure will occur below the Geopier element within the underlying matrix soils. A Geopier foundation system consisting of individual Geopier elements widely spaced along a lightly loaded strip footing within a soft clay soil is an example of a condition that might result in this type of bearing capacity failure. It is pointed out that the Geopier element, even in soft soils, has a bottom bulb, which will provide additional resistance to punching through matrix soils.

The third type of failure, and that considered to be the most probable situation for most subsoil conditions, including granular soils and soft to stiff fine-grained soils, is for a **composite punching** failure of the composite zone itself through the in-situ soil (Figure 4.3c). Punching resistance in this case would occur within the in-situ soil adjacent to the vertical perimeter surface of the composite zone. A general shear failure would then have to occur within the in-situ soils underlying the composite Upper Zone.

It can be seen that the two methods of punching failure are similar, but not identical. **Individual Punching** (Figure 4.3b) assumes that **individual piers** will punch through adjacent matrix soils. **Composite Punching** (Figure 4.3c) assumes that the composite Upper Zone containing a group of piers will punch through adjacent matrix soils as a composite mass. In each of these situations, general shear failure must also occur in the soils underlying the individual Geopier elements or underlying the composite Upper Zone respectively.

Lawton (1995) provides a more detailed discussion of these analyses, including appropriate equations. To develop bearing capacity calculations for Geopier soil reinforcements, passive soil pressures should be used to model the horizontal stresses within the matrix soils adjacent to Geopier elements. These stresses are perpendicular to the vertical perimeter Geopier surface.

The ultimate bearing capacity of a Geopier foundation will be greater than that of the matrix soil alone because the reinforced Upper Zone will deepen the zone within which the failure occurs. This significantly increases the effective depth of the footing (the interface plane between Upper Zone and Lower Zone becomes the new "bottom of the footing"), and hence, the punching resistance for both cases discussed above. In addition, build-up of lateral stresses within matrix soils from the installation of Geopier foundations will widen the effective diameter of the Geopier elements and widen the effective composite zone. Each of these situations results in an increase in ultimate bearing capacity of the Geopier-supported foundation system.

In general, a minimum factor of safety of 3.0 against ultimate bearing capacity failure is recommended for footings supported by Geopier elements. If the factor of safety is less than 3.0, increasing the length of the Geopier element in order to deepen the composite Upper Zone will increase the factor of safety for bearing capacity failures due to punching. Placing Geopier elements outside of the footing perimeter will further result in increasing the factor of safety against ultimate bearing capacity failure. Perhaps the simplest method for increasing the factor of safety against ultimate bearing capacity failure is to reduce the composite bearing pressure of the footing by increasing the footing's plan dimensions. In addition, it should be noted that the composite bearing pressures for isolated footings, listed on Table 4.2, will provide a factor of safety against ultimate bearing capacity failure which is greater than 3.0.

A series of design examples are provided below which provide a range of values for bearing capacity safety factors for different footing sizes, bearing pressures and failure types.

Three different sized footings supported by Geopier elements are used to provide example comparisons of ultimate bearing capacities with the same subsoil conditions. Footing dimensions and Geopier element sizes and spacing are listed below (two different Geopier shaft lengths are considered for the 8.5-ft. square footing):

Case	Square Footing Dimension, ft	Number of Piers	Pier Diameter & Shaft Length	Spacing, Edge-to-Edge, inches
A	6 ft. x 6 ft.	Four	30 inch, 6 ft.	12 inches
B	8.5 ft. x 8.5 ft.	Nine	30 inch, 7 ft.	9 inches
C	8.5 ft. x 8.5 ft.	Nine	30 inch, 9.5 ft.	9 inches
D	11.25 ft. x 11.25 ft.	Nine	30 inch, 10 ft.	18 inches

Given:

Soil Conditions: Residual sandy silts and silty sands with N values of 7 to 10 in the Upper Zone and N values of 10 to 15 in the Lower Zone. Allowable bearing pressure based on settlement for footings supported on the matrix soil alone is 2500 psf, and the effective friction angle of matrix soils is 28 degrees. The effective friction angle of the composite Upper Zone with Geopier elements is 45 degrees.

k_{gp} = 260 pci
k_m = 17.5 pci

Using the method outlined in Lawton (1995) for the condition of **Composite Punching** (Figure 4.3c) of the Upper Zone through the in-situ soil, the following ultimate bearing capacities and loads were calculated:

Case	Ultimate Bearing Capacity (ksf)	Ultimate Load (S.F. = 1.0) (kips)	Max. Bearing Pressure (S.F. =3.0) (ksf)	Ultimate Load (S.F. = 3.0) (kips)
Case A	20.4	734	6.8	245
Case B	32.8	2,370	10.9	790
Case C	36.3	2,623	36.3	874
Case D	42.6	5,391	42.6	1,797

From these calculations, Figure 4.3.1 was developed showing factors of safety for the three different footing sizes.

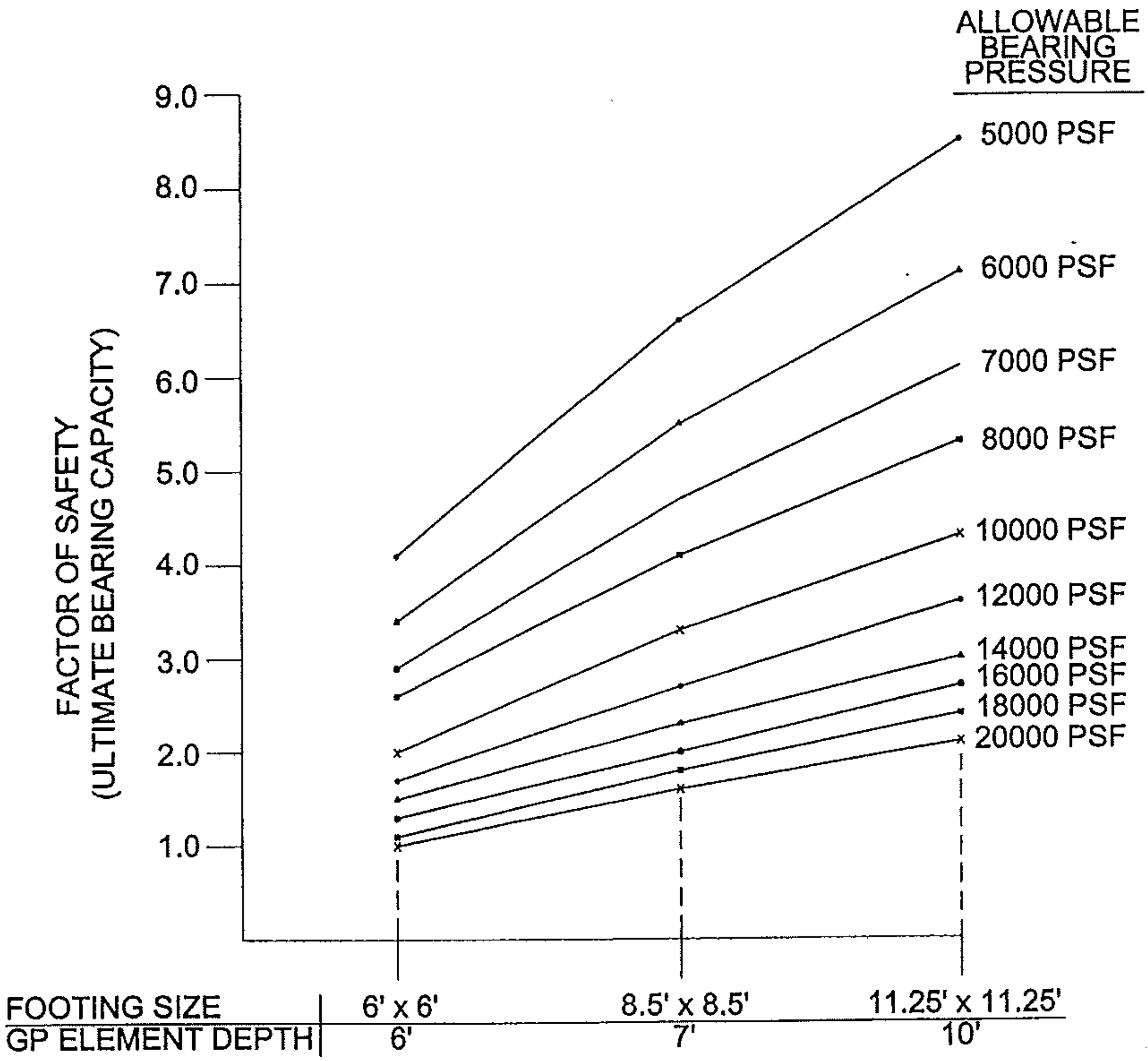
As a further example, safety factors for the four cases are shown below for a column load of 850 kips. We point out that these bearing capacity calculations are not directly applicable to design. Design analyses of Geopier-supported foundations require settlement estimation calculations.

Case	Footing Bearing Pressure with 850 kip Load	Factor of Safety against Ultimate Failure
A	23,611 psf	0.86
B	11,765 psf	2.79
C	11,765 psf	3.09
D	6,716 psf	6.34

These results for Case B (7 foot shaft length) and C (9.5 foot shaft length) for the same sized footing and same load, point out the influence of Geopier element shaft length on increasing the factor of safety for bearing capacity.

For the mode of bearing failure occurring within the Geopier **Upper Zone**, Figure 4.3a, a condition that can exist if the Lower Zone is extremely hard or dense and the bearing pressure is high, one may use classical bearing capacity theory. It seems reasonable that the bearing capacity of mode (a) should exceed that of mode (c), if the Lower Zone value for shear strength is not extremely high.

Calculations were made to determine the safety factor against an Upper Zone bearing capacity failure (Figure 4.3.a) for the above four cases using the method described in Handy, 1982. The depth of the failure zone within the Geopier element below the bottom of footing was assumed to be equal to 5 feet. The results listed below show that a significantly higher factor of safety is obtained for potential failure within the Geopier elements (Upper Zone).



Note: Charts are based upon Geopier Elements founded in Matrix Soils with an effective Friction Angle of 28°

Figure 4.3.1 Bearing Capacity FOS Charts (Ø = 28° Condition)

Case	Ultimate Bearing Capacity	Footing Bearing Pressure with 850 kip Load	Factor of Safety against Ultimate Failure
A	43,611 psf	23,611 psf	1.85
B	51,294 psf	11,765 psf	4.40
C	51,294 psf	11,765 psf	4.40
D	59,797 psf	6,716 psf	8.90

Therefore, for application where a hard Lower Zone does not exist, a composite punching failure mechanism will govern the bearing capacity safety factor. However, both Upper Zone and composite punching should be checked when a hard or dense Lower Zone exists to see which mechanism controls the safety factor.

4.4 Geopier Intermediate Foundation Uplift Capacity

Geopier intermediate foundation systems have been used since 1991 as uplift anchors during modulus load tests. The uplift load is transferred by means of steel plates placed at the bottoms of Geopier elements. Threaded rods or bars transfer the uplift loads to the load test frame system of beams. Initially, when uplift load tests were performed, the uplift resistances observed seemed surprisingly high. Furthermore, in non-clayey soils Geopier uplift elements exhibited a near-elastic behavior. Rebounds were measured to be 100%, or nearly 100%, in most cases for Geopier uplift anchors within non-clayey soils.

A simplified uplift calculation method is used based upon a theoretical failure plane along the vertical surface of the Geopier element/matrix soil (Figure 4.4.1). The normal stress (σ_N) is applied to the planar area, and the uplift capacity is calculated using the classic shear strength formula of $S = \sigma_N \times \text{Tan } \phi'$. The estimate of "$\sigma_N$", the normal stress perpendicular to the uplift shear force, is assumed to be the passive pressure limit of the soil. The value of ϕ', the friction angle of the matrix soil, should normally be the drained shear strength of the soil. A reasonable safety factor should then be applied. It is recommended that a minimum safety factor of 2.0 be used with this method. A safety factor of 1.5 may be applied to results of full-scale uplift test on Geopier elements.

A more rigorous theoretical analysis of the uplift capacity of Geopier short aggregate piers is provided in Lawton, Fox, and Handy, 1994.

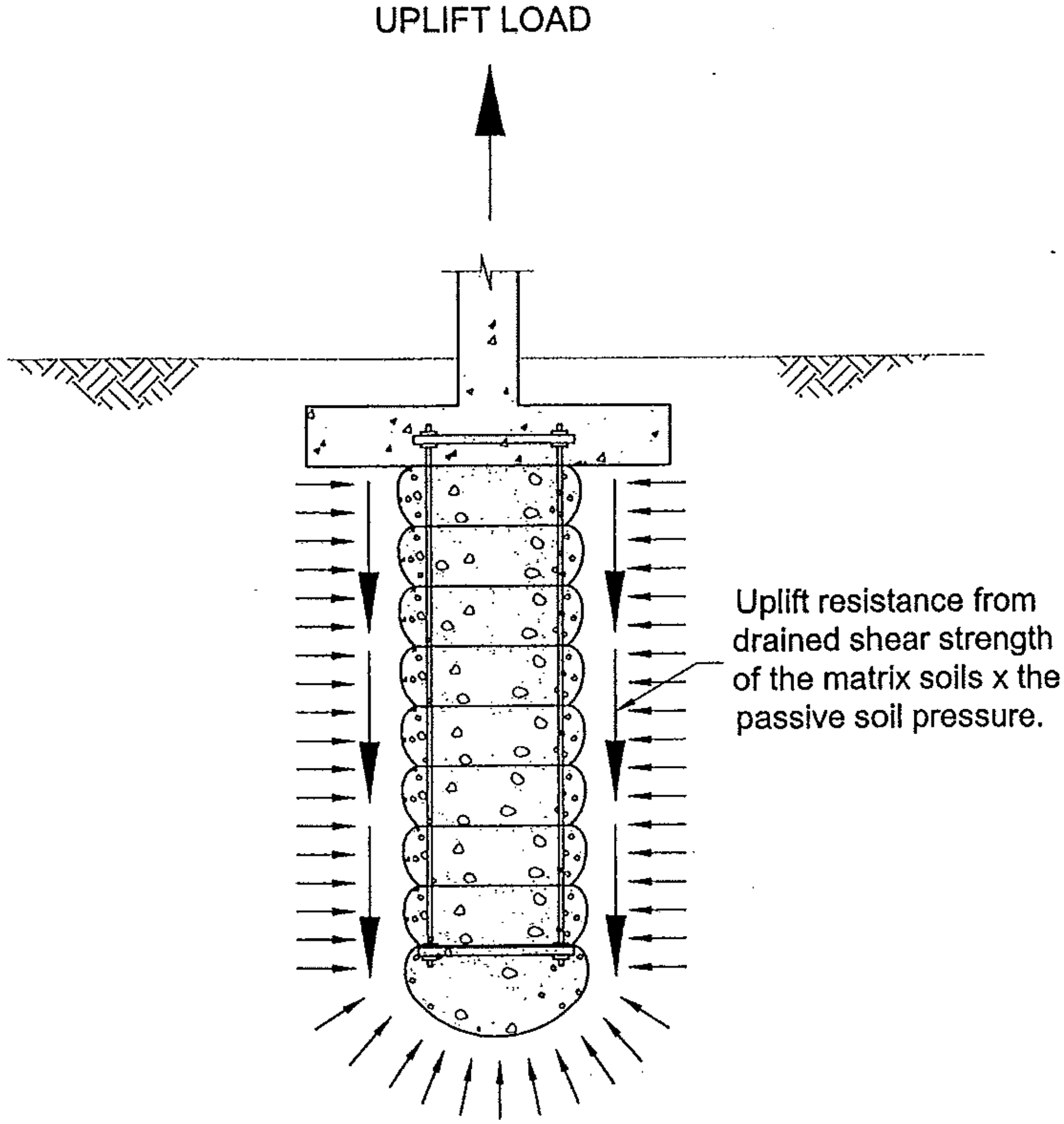


Figure 4.4.1. Uplift Load Resistance

4.5 Soil Reinforcement

4.5.1 Geopier Element Lateral Load Resistance

The lateral load capacity of a footing supported by Geopier elements to resist sliding resistance at the footing bottom/Geopier element interface is determined in the following manner:

The stress concentration on top of the Geopier elements (q_{gp}) is estimated **from dead load only.** This stress intensity times the footprint area of Geopier elements, represents the normal force (σ_N) acting perpendicular to the shear force. The shear resistance, or lateral resistance to sliding equals the calculated normal force times tangent of ϕ', where ϕ' is the drained friction angle of the densified graded aggregate in the Geopier element. For base course stone, the friction angle from actual testing is 52.2 degrees, which results in a tan ϕ' of 1.29. For open-graded #57 stone, the friction angle from actual testing is 48.8 degrees, which results in a tan ϕ' of 1.14

The stress concentration on Geopier elements is determined from the stiffness ratio of the Geopier element to the matrix soil. This has varied (on over 175 projects) from about 8 to about 32, with most soils showing a stiffness ratio of 10 to 15. Because the method of estimating matrix soil stiffness contains an inherent built-in safety factor, the estimate may be made more accurate and more conservative by multiplying the matrix soil modulus by 1.5. For greatest accuracy, use actual load test data for the modulus of the Geopier element.

Using actual load test data for the modulus of the Geopier element, it is recommended that a safety factor of 2.0 be used in calculating the available lateral load resistance. Without load test data, and using estimated Geopier modulus values, a safety factor of 2.5 is recommended.

Using the example started in Section 4.2.1, an example is presented below:

Given:

Soil Conditions: Sandy silt matrix soil with an allowable bearing pressure of 3000 psf and an average N value within the Upper Zone of 9. Lower Zone soils consist of Piedmont Region residual sandy silts with an average N value of 10.

Column Load: 600 kips

Friction Angle of Geopier Element = 52 degrees

From the previous example (4.2.3):

q = 7000 psf from Table 4.2
k_{gp} = 260 pci from Table 4.2

k_m = 3000 psf /144 sq. in/sf / 1 in = 20.8 pci
R_s = 260 pci/20.8 pci = 12.5
q_m = 1460 psf
$q_{gp} = R_s\, q_m$ = 12.5 x 1460 psf = 18,250 psf
A = 9.25 x 9.25 = 85.6 sf

Calculate the Lateral Load Resistance

Geopier Capacity from Table 4.2 = 105 kips

Number of Geopier Elements Required = 600/105 = 5.7 → **6 Geopier Elements**

Given that 60% of column load is dead load.

Calculate the vertical stress from the footing dead load on the Geopier element (q_{gp})

R_a = 6 x 4.9/85.6 = 0.34
q = 0.60 (600,000)/85.6 sf = **4,203 psf**
$q_{gp} = q \cdot R_s/(R_a \cdot R_s - R_a + 1)$
$\mathbf{q_{gp}}$ = 4,203 psf x 12.5/(0.34 x 12.5 - 0.34 + 1) = **10,700 psf**

Calculate the Geopier Foundation Area:

$\mathbf{A_{gp}}$ = 6 x 4.9 sf = **29.4 sf**

Lateral Load Resistance = $L_{gp} = q_{gp} \times A_{gp} \times \tan\phi_{gp}$

Where $\tan\phi_{gp}$ = tan 52° = 1.28

$\mathbf{L_{gp}}$ = 10,678 psf x 29.4 psf x 1.28 = 402,662 lbs = **403 kips**

Using a factor of safety of 2, the lateral load resistance for design becomes **201 kips.**

If the sliding resistance of the soil is considered:

$L_m = q_m \times A_m \times \tan\phi_m$

For silty soils, the tangent interface friction angle is conservatively estimated at 0.35, therefore:

q_m = (4203 psf/12.5) = 336 psf

L_m = 336 psf (85.6 – 29.4)(0.35) = 6609 lbs. = 6.6 kips

Using a factor of safety of 2, L_m for design becomes

L_m = 3.3

Therefore, the total lateral load resistance from the Geopier elements and soil is:

$L_{TOTAL} = L_{gp} + L_m = 201 + 3.3$

L_{TOTAL} = 204 kips

Normally, sliding resistance provided by the matrix soil is disregarded since it is small in comparison with sliding resistance provided by Geopier elements.

4.5.2 Geopier Elements for Slope Stabilization

Geopier soil reinforcement can provide designers with another tool for slope stabilization. Geopier elements have been utilized for stabilization of slopes beneath retaining walls, including segmental walls and cast-in-place walls, as well as for stabilization of active landslides.

The mechanics involved in using Geopier elements for slope stabilization are simple modifications of existing limit equilibrium theories. By removing a mass of weak soil and replacing it with highly densified aggregate with significantly greater shear strength, stability is increased. In addition, the lateral stress build-up in the matrix soils surrounding aggregate piers will tend to increase the shearing resistance of the matrix soils above that which they would exhibit under normal in-situ pressures.

When Geopier reinforcements are used at the toe of unstable slopes, the greater unit weight of the aggregate piers provides additional stability. The overall effect of the improvement is the creation of a large, stabilized mass of soil that can be reasonably modeled using methods discussed below.

The current state-of-practice for modeling slopes reinforced with Geopier elements is to use the average shear strength method (Barksdale, 1983) combined with computer slope stability analysis programs such as PCSTABL. The computer model considers the Geopier soil reinforcement zone as a homogeneous block of soil that can be entered into a data file. The shear strength for the reinforced zone is greater than the unreinforced zone. However, this approach does not consider the significant stress concentration, and hence higher shear strength that can occur for Geopier elements placed beneath fills or wall loads. The average shear strength method, without considering the effect of stress concentration on shear strength, is often over-conservative and can adversely affect the economics of a project.

To account for the vertical stress concentrations in computer slope stability analyses, Barksdale (1983) recommends a method whereby very thin, fictitious strips of high density, zero shear strength soil are placed above the aggregate piers and low density, zero strength soil strips are placed above the matrix soils. The density used for the strips is related to the relative stiffness of the Geopier elements to the matrix soils. With respect to data entry, this method is considerably more cumbersome than the average shear strength method. To use the fictitious strips, a profile must be modeled that considers each row of aggregate piers as an equivalent, continuous strip. The required data entry can become excessively tedious even when making subtle refinements to the Geopier layout and the computer model. Alternatively, hand calculations utilizing the added shear strength from stress concentrations on Geopier elements can be performed on the critical failure surface. While this method may not provide the lowest factor of safety for a given project, it will provide the user with a better idea of the actual safety factor.

Slope Stability Analyses

To design Geopier soil reinforcement for a slope stability problem, first the unreinforced condition is modeled and analyzed to determine the scope of the problem. As with all slope stability models, it is critical that slope geometry, external loading and subsurface conditions be as accurate as possible. It may be necessary to perform a sensitivity analysis on the model to determine what data are most critical and require the greatest attention to detail. For stabilization of active landslides, the location of the failure surface and shear strength of materials in the failure zone should be known in order to develop an appropriate remediation plan.

When the unimproved conditions have been defined, it is then a trial and error process to determine the Geopier element quantity, diameter, depth and spacing pattern needed to provide the most practical solution. Because of the iterative nature of the analysis, it is simplest to use the average shear strength method for reasons discussed above. The designer may choose to check the critical failure surface by hand using the higher shear strength as a result of stress concentrations on the Geopier elements to better refine the model and design. Finally, the model should be checked for appropriate factors of safety.

An important consideration for Geopier reinforcement for any slope stabilization technique is construction sequence. It may be necessary to temporarily alter the slope to gain access with construction equipment, thereby creating temporary instability. If there is a significant risk of localized failure resulting from temporary construction, it will be necessary to assess the stability of the short-term condition. In such cases where unacceptable temporary instability exists, care should be taken in developing a sound construction schedule.

To determine the weighted average shear strength for a Geopier reinforced soil zone, without considering any stress concentration on the Geopier elements, the following equations are used.

$$\gamma_{avg} = \gamma_{gp} R_a + \gamma_m (1-R_a) \qquad \textbf{Eq. 11}$$

$$\tan\phi_{avg} = \frac{\gamma_{gp} R_a \tan\phi_{gp} + \gamma_m (1-R_a) \tan\phi_m}{\gamma_{avg}} \qquad \textbf{Eq. 12}$$

Where:

γ_{gp} = Unit weight of the Geopier element = 145 pcf
γ_m = Unit weight of the matrix soil
γ_{avg} = Average unit weight of the reinforced soil (Eq. 11)
$\tan\phi_{avg}$ = Tangent of the average effective friction angle for the reinforced soil.

Using these equations the following example can be computed.

Given:

A shallow landslide in residual silty soils has occurred and Geopier soil reinforcements are being considered to reinforce the existing slope. An initial equilateral triangular spacing of 4 rows of 30-inch diameter Geopier elements at a 3.5-foot center-to-center spacing perpendicular to the potential failure surfaces (Figure 4.5.2.1) is used.

The area ratio can be calculated for a given equilateral spacing as follows:

$$R_a = \frac{n \times A_{gp}}{[(n-1)\, s\, (0.866) + D]\, s} \qquad \textbf{Eq. 13}$$

Where:

s = Center-to-center spacing of Geopier elements
n = Number of rows of Geopier elements
D = Diameter of a single Geopier element
A_{gp} = Area of a single Geopier

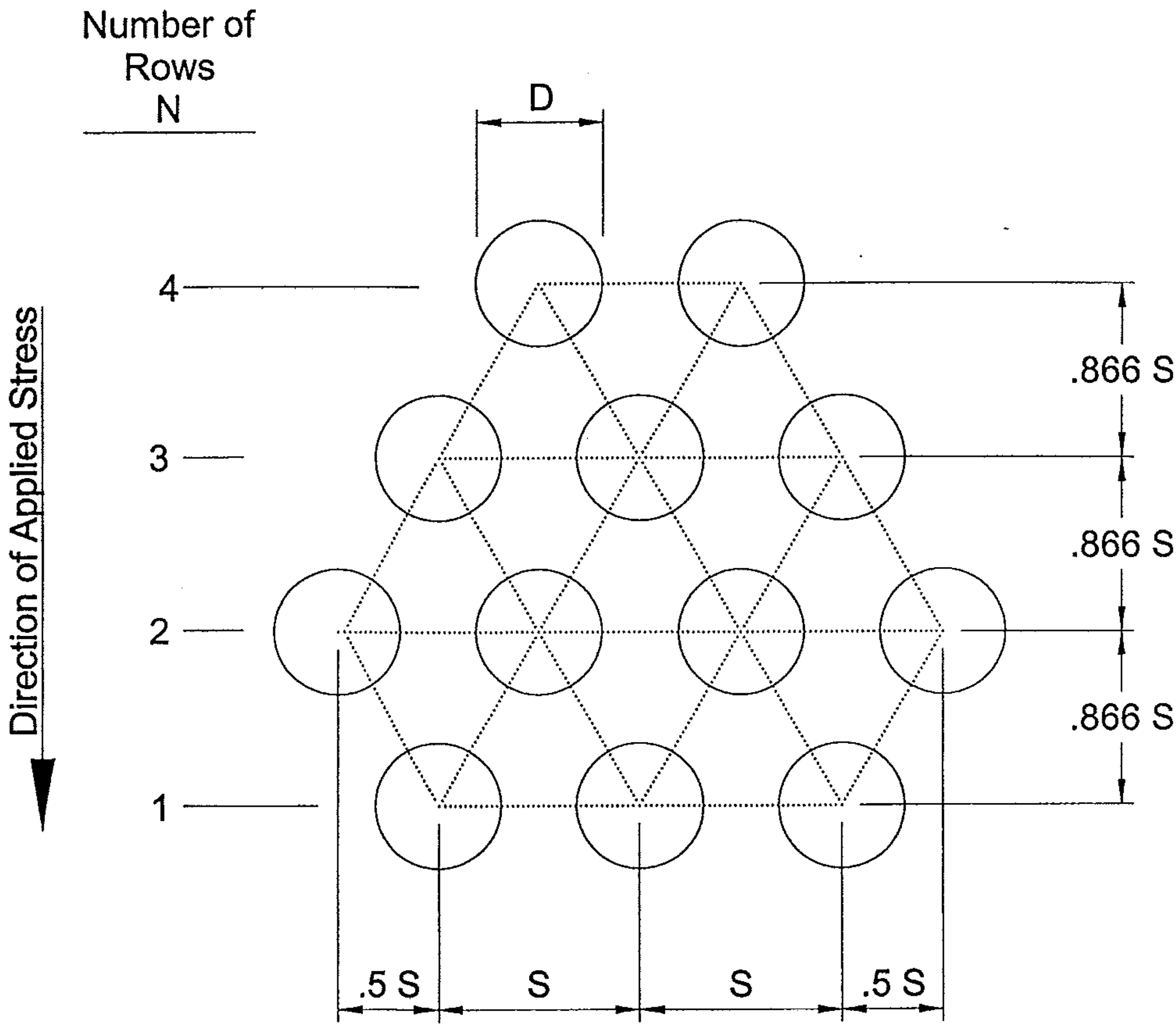
$$R_a = \frac{4 \times 4.91\text{sf}}{[(4-1)\, 3.5\, (0.866) + 2.5]\, 3.5} = 0.483$$

The soil and Geopier element properties for the project are as follows:

γ_{gp} = 145 pcf
γ_m = 120 pcf
R_a = 0.483
ϕ_{gp} = 52°
ϕ_m = 28°

Using Equation 11 the average unit weight, γ_{avg}, of the reinforced soil is calculated as follows:

γ_{avg} = 145 pcf (0.483) + (120 pcf)(1-0.483)
γ_{avg} = 132.1 pcf



*Min. Spacing for S = D+1'-0"

Figure 4.5.2.1 Equilateral Triangular Spacing For Geopier Elements For Area Stabilization

Using Equation 12 the weighted average shear strength is calculated as follows:

$$\tan\phi_{avg} = \frac{145 \text{ pcf } (0.483) \tan(52°) + 120 \text{ pcf } (0.517) \tan 28°}{132.1 \text{ pcf}}$$

$\tan\phi_{avg} = 0.9283$

$\phi_{avg} = 42.9°$

Using the weighted average shear strength slope stability, analyses can be performed using a Geopier reinforced fill zone. Figure 4.5.2.2 shows a typical cross section of a stability analysis performed using a Geopier reinforced soil zone (**Gpzone**) placed beneath a retaining wall as part of a landslide correction. Note that this analysis does not consider the stress concentration that will exist from the retaining wall surcharge. Given a 10:1 stress ratio, the normal force within the Geopier element will actually be approximately 2 times higher than that used in the analysis. The actual safety factor will increase from 1.35 with the stress concentration taken into account.

4.6 Geopier Foundation System Layout Considerations

In addition to the allowable composite footing bearing pressure, several factors need to be considered when sizing footings. These include:

- Size of Geopier Element – It is important that the Geopier foundations fit under a footing with enough room between Geopier elements to make installation practical. Typically, the practical minimum edge-to-edge distance between Geopier elements is 12 inches.

- Layout of Geopiers Elements – Depending on the number of Geopier-supported footings required, a rectangular layout is sometimes preferred over a square footing, to minimize concrete and optimize spacing. This is particularly true when there are two Geopier elements supporting a footing.

It should be noted that the actual zone of influence of a Geopier element is much larger than the drill depth diameter. Geopier soil reinforcements provide an improved subgrade on which a spread footing can be founded and settlement minimized. Finite element analyses have shown that location of Geopier elements outside and partially outside the edges of footings can improve performance. Therefore, spacing guidelines for spread footings with Geopier element areas of 30% or more may reflect construction limitations in terms of spacing, not performance requirements. Likewise, placement of Geopier elements outside or partially outside the footprint of a footing is acceptable practice within limitations of experience and special analyses. This has seldom been done in practice.

Figures 4.6.1 and 4.6.2 provide typical spacing guidelines and typical layouts for Geopier elements for a number of column and continuous footing conditions.

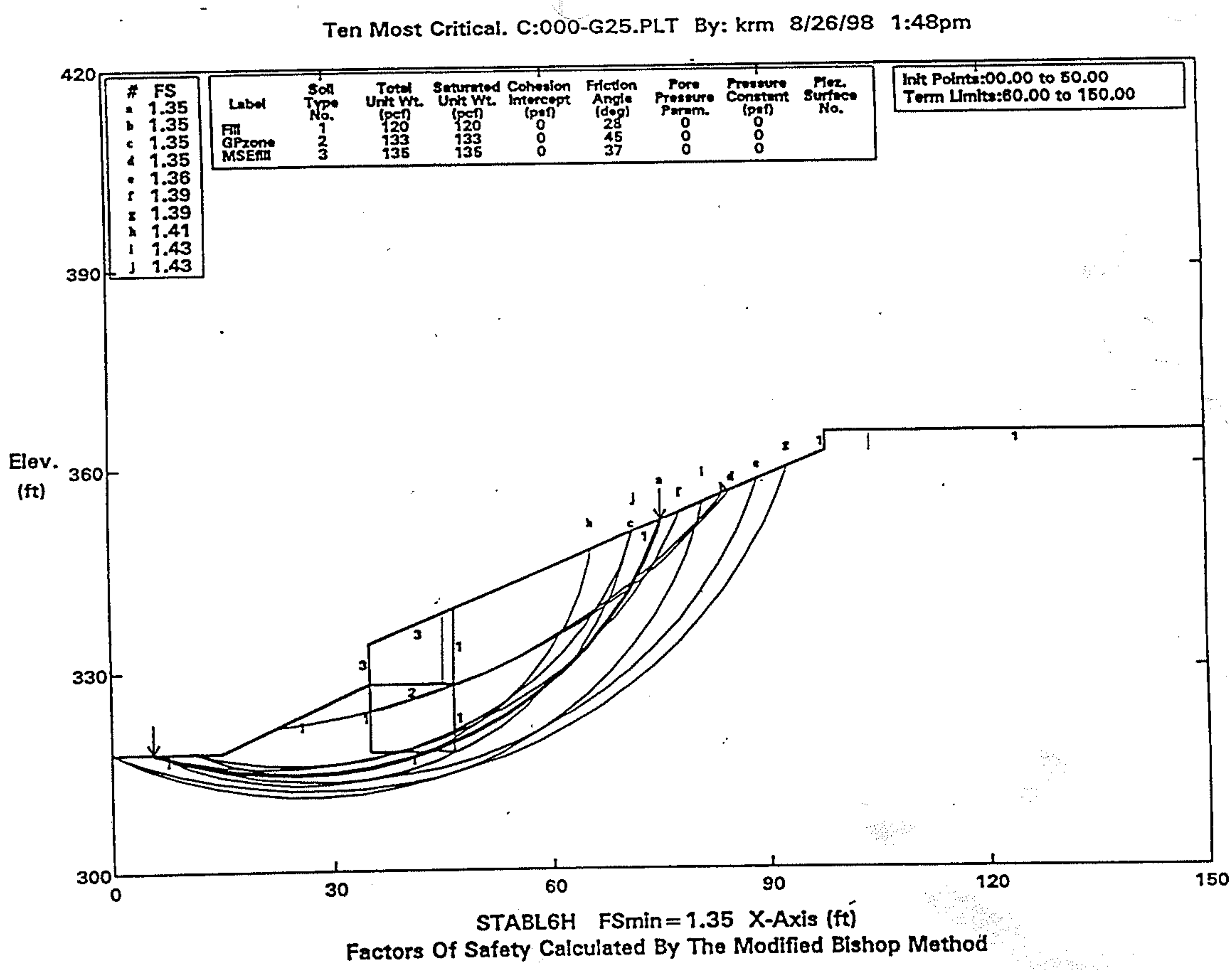
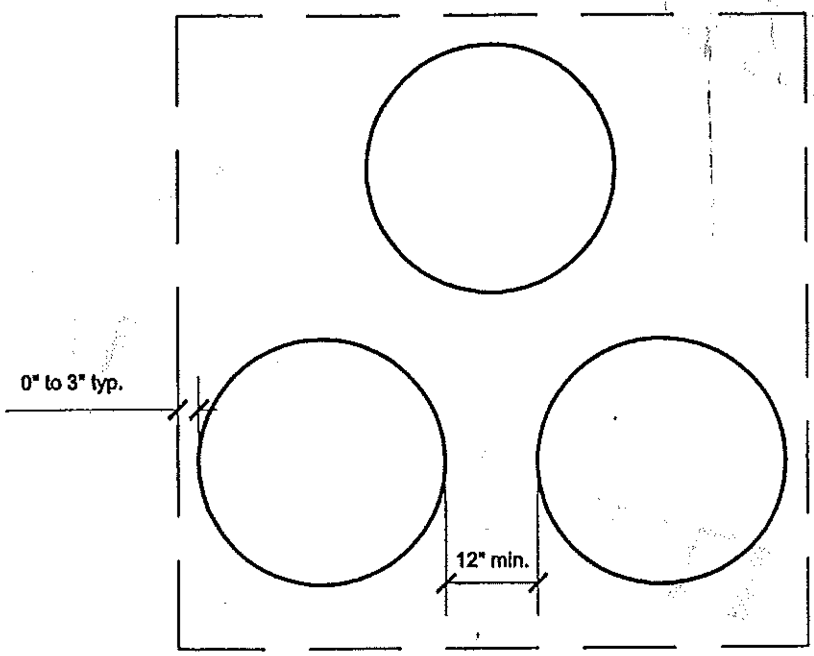
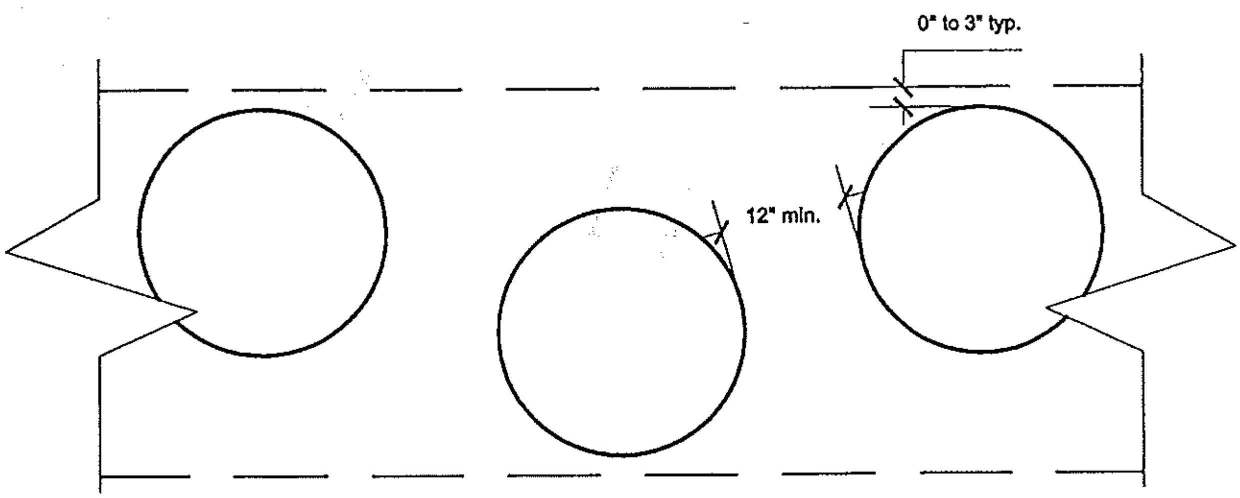


Figure 4.5.2.2 Factor of Safety Calculations using The Modified Bishop Method



Isolated Column Footings



Continuous Strip Footings

Figure 4.6.1. Spacing Guidelines for Geopier Elements

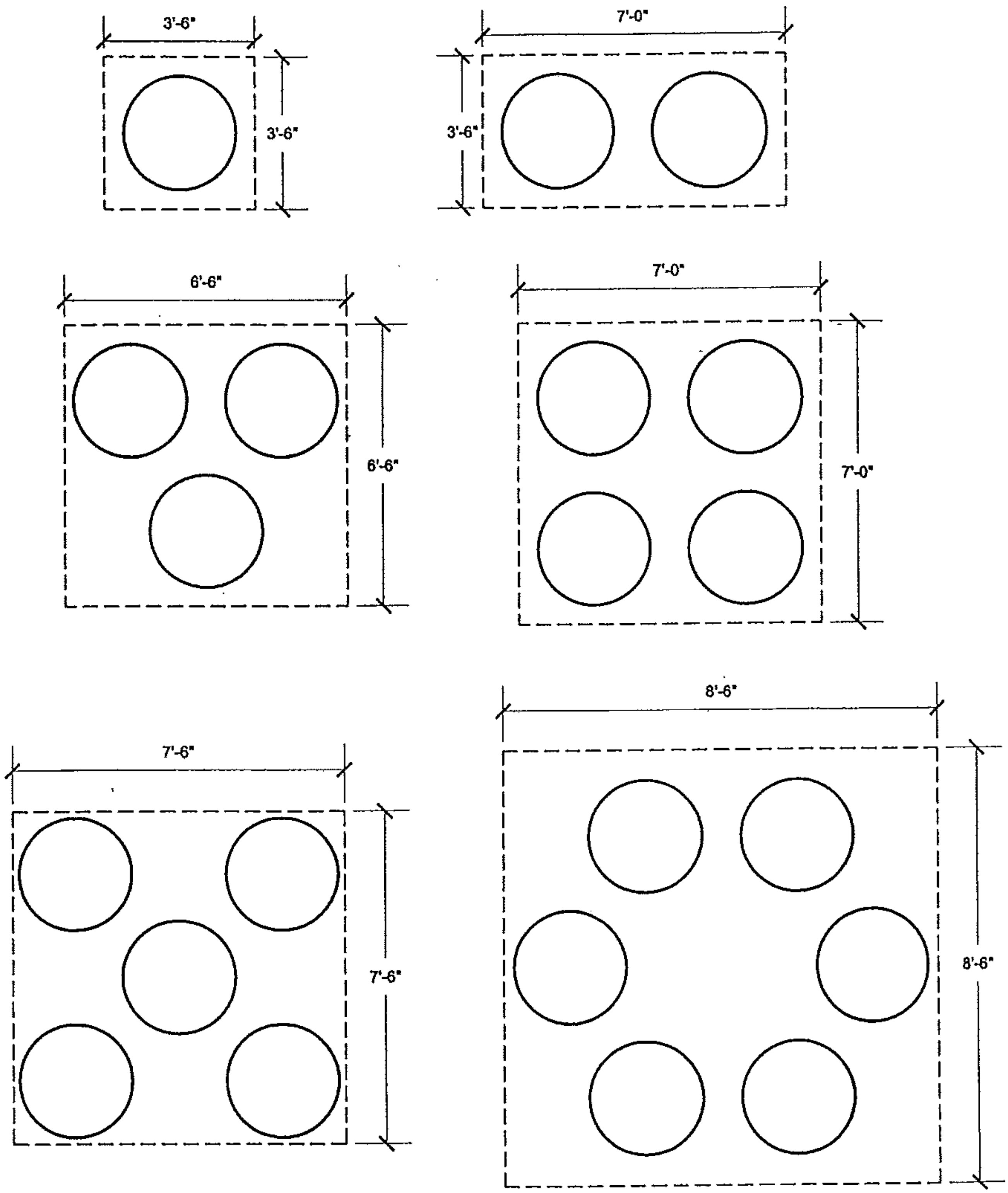


Figure 4.6.2. Typical 30" Dia. Geopier Element Layouts for Selected Footings

5.0 GEOPIER FOUNDATION CONSTRUCTION

Applying geotechnical judgment requires making assumptions regarding the subsurface conditions at the site. The easy way out is to assume the worst *possible* conditions and design from there, but that is not always economical. A more progressive approach involves designing on the basis of the worst *probable* conditions, while understanding that less-favorable conditions may exist. This approach requires proactive construction methods, including monitoring of site conditions during earthwork operations, and defining means and methods of dealing with variance from design assumptions. This process has been commonly referred to as the *observational method*, and it is by this method that Geopier intermediate foundations are designed and constructed.

When the design process is complete, the real work begins. Designing a Geopier-supported foundation based on geotechnical reports and test boring records is a relatively simple, straightforward process. The initial design work is performed in a relatively pressure-free environment with little financial risk; no expensive equipment is mobilized, no materials are purchased, and no labor costs are accrued. The challenge arises when, upon installing Geopier foundations, site conditions are less favorable than were reasonably assumed from available subsurface information. Gross variations in subsurface conditions immediately become apparent during drilling and tamping operations. More subtle differences may become apparent after a modulus load test is conducted and the data are analyzed. These events do not occur until cash is already flowing and getting everybody's attention. A proactive approach to construction, such as that described in this chapter, ensures that these real-world challenges are met, and the Geopier elements do their job.

5.1 Preconstruction Planning

Successful Geopier foundation projects, just as with other types of construction, rely on thorough planning and coordination. This is not always an easy task, as there may be several separate engineers or architects involved in the design process. Further difficulty can arise when coordinating the various trades, ranging from excavation and site work to concrete subcontractors, which may be involved in the project. Providing Geopier reinforcements that meet project requirements may include input from other design professionals and trades that are involved in the project. Whether assisting the Structural Engineer in determining design loads and preparing footing details, monitoring load tests and installation for the Geotechnical Engineer, or coordinating footing excavation and preparation with the concrete sub-contractor, everyone involved must understand how they contribute to successful installation of Geopier elements.

5.2 Geopier Element Load Testing

One common feature of nearly all structures, whether Geopier-supported or not, is that they must rely on subsurface information that is often only qualitative, or at best contains specific soil boring data spaced at fairly great intervals. Geotechnical investigation and testing procedures have been developed over the years, and the information obtained from these methods has been used for design. In some cases test boring records are all that is needed for design. In other cases, specialized laboratory or in-situ field testing may be necessary. In either situation, modulus load tests on Geopier elements can provide essential confirmatory information on Geopier element stiffness and behavior of the Geopier element under load within the Upper Zone.

Most Geopier foundation projects include either a modulus load test or an uplift load test, or both. On occasions, such as when two significantly different subsurface conditions exist on a site, two (or more) modulus load tests are performed. A wealth of information has been generated over the last ten years of Geopier load testing. It is now possible to predict with reasonable accuracy the characteristics of the Geopier element and the matrix soil from available test boring records for a wide variety of subsoil conditions. In fact, it is typically required to do so in order to prepare a design and a bid. Since subsurface conditions can vary significantly from the information shown on test boring records, and because of limitations in the data and knowledge of subsoil engineering characteristics and past stress history of subsoils, modulus load testing is performed to verify design assumptions.

When loads are light, soil conditions are well known, structural uncertainties are minimal, and the number of Geopier elements required for a project is relatively small, load testing may not be required. Under such circumstances there may not be sufficient value added in installing and performing a load test. In such cases, Geopier foundations are designed with an added level of conservatism.

5.2.1 Modulus Load Testing of Geopier Elements

The Geopier element stiffness modulus is determined by applying downward pressure to the top of a Gèopier element in a series of load increments, which are determined from design calculations. When a Geopier-supported foundation is designed, it is necessary to determine the stresses on each Geopier element, which can then be used to predict Upper Zone settlements. Since footing loads often vary from one footing to the next, the design Geopier stress also differs. The load increments used in the test should be based on the maximum stress on the Geopier element calculated for the project. Once the maximum stress is determined, the load increments can be calculated according to the schedule shown in Table 5.2.1. To convert the stress on the Geopier element to a jack load, simply multiply the maximum stress by the plan area of the Geopier element (= 3.14 ft^2 for 24", 4.91 ft^2 for 30", and 7.07 ft^2 for 36" diameter Geopier elements).

Table 5.2.1. Typical Modulus Load Test Schedule

Increment	Approximate Stress on Geopier Element (% of maximum) design)	Minimum Duration, Minutes	Maximum Duration, Minutes
Seat	< 9	N/A	N/A
1	17	15	60
2	33	15	60
3	50	15	60
4	67	15	60
5	83	15	60
6	100	15	60
7	117*	60	240
8	133	15	60
9	150	N/A	N/A
10	100	N/A	N/A
11	66	N/A	N/A
12	33	N/A	N/A
13	0	N/A	N/A

* = longer load increment

Loads are applied to the top of the Geopier element using a hydraulic jack and load frame as shown in Figure 5.2.1. At each load increment, the deflection is measured using at least two dial gauges accurate to 0.001 inches, and the gauge readings are recorded. Loads are held at least for the minimum duration shown on the schedule. The load is maintained until the rate of deflection is less than 0.01 inches per hour (0.0025 inches per 15 minutes) or until the maximum time duration is reach, whichever occurs first. The amount of deflection at a given load increment is equal to the average of the last dial gauge readings taken for that load increment minus the average of the dial gauge readings taken at the seating load. The deflection for each load increment is then plotted against the stress for that increment. The modulus used for design is equal to the design stress divided by the corresponding deflection at that stress. The modulus is then used for estimating Upper Zone settlements. Examples of a completed load test form and a Stress vs. Deflection curve are shown in Figures 5.2.2 and 5.2.3.

This procedure is based on portions of ASTM D 1143 and ASTM D 1194. As shown in the table, the maximum load applied during the modulus load test is typically equal to 150% of the maximum design stress. The ASTM procedures *for piles* require loads up to *200% of the maximum design load*. This is because pile load tests are performed primarily to determine the *pile bearing capacity*, and therefore, requires a Safety Factor of 2.0. The Geopier modulus load test is not performed to determine bearing capacity,

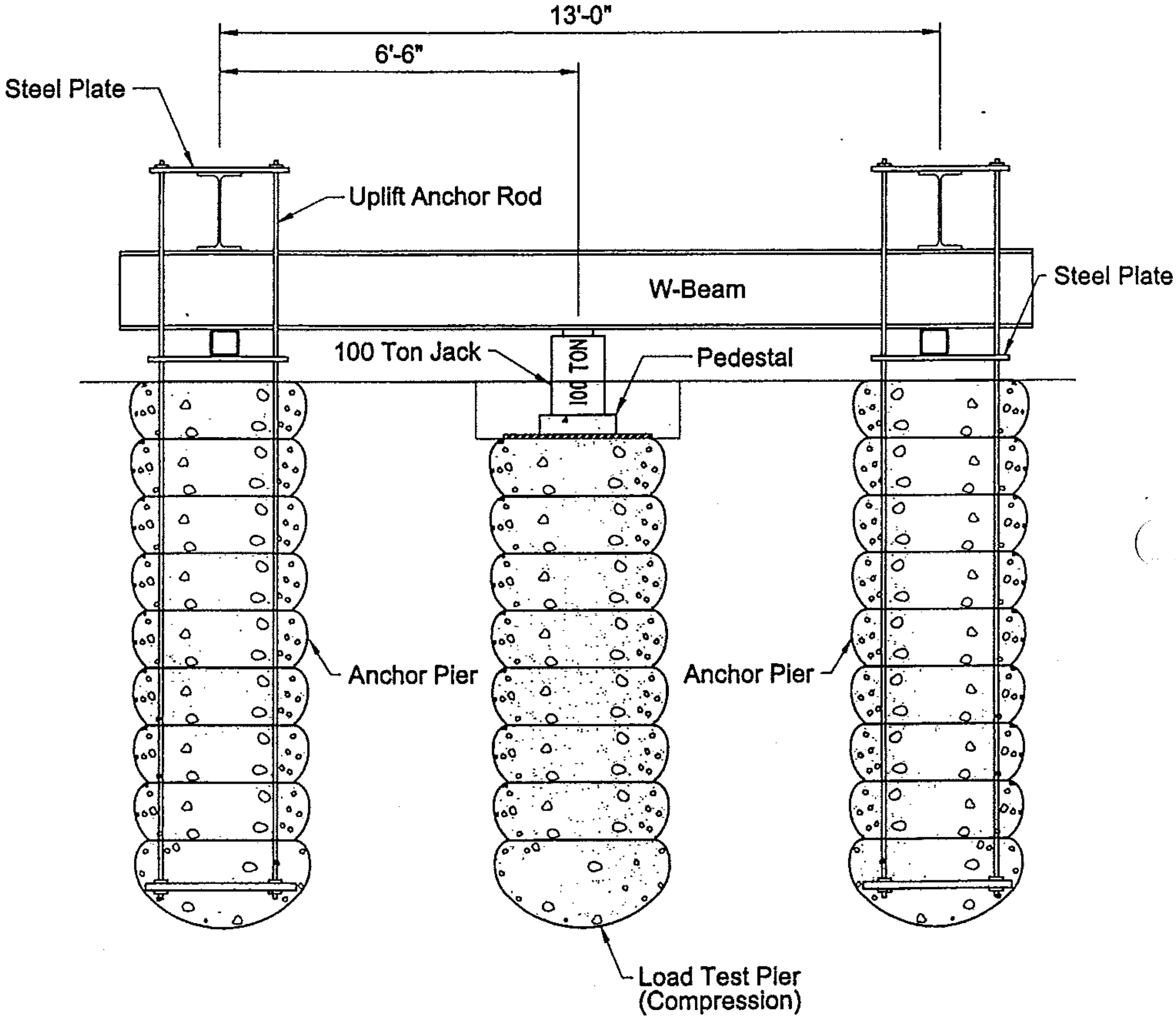


Figure 5.2.1a. Typical Modulus Load Test Setup

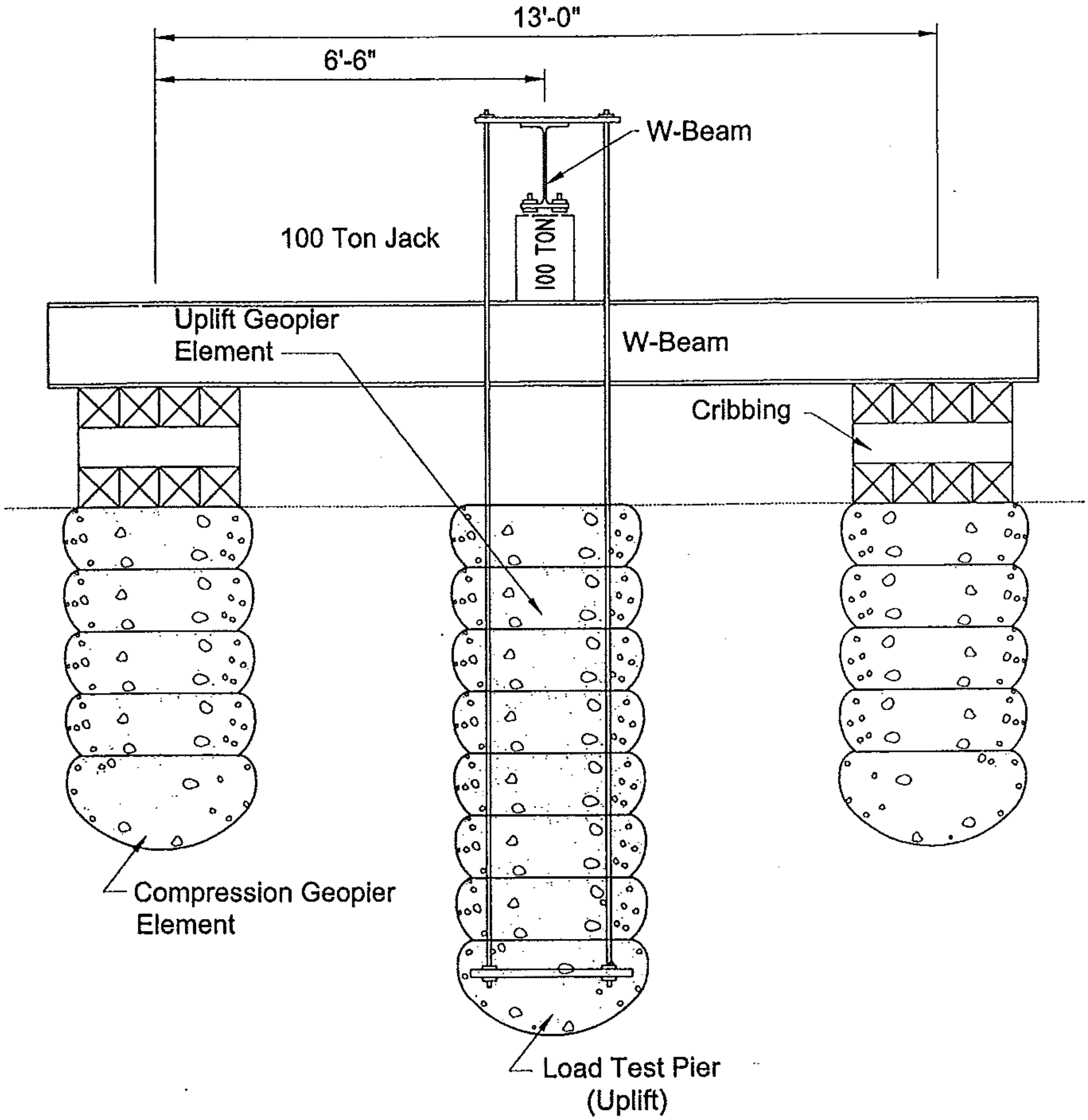


Figure 5.2.1b. Typical Uplift Test Setup

geopier ™

Geopier Modulus Load Test Data

Project: Geopier Foundation Manual Example

Design Stress (psf): 10,000 P Dia.(in.): 30 Depth(ft): 8 Install Date: 4/16/98 Test Date: 4/20/98

Adjacent Boring(s): B1, B4 and B7 logged by Geotechnical Consultant, 1/12/98; GW depth approx. 12 ft below grade Weather: Sunny, 70°

General Req'ments: max. test stress = 150% max. design stress; GP modulus, K_{gp} = 200 pci, for deflection, δ = 0.35 in. @ 10,000 psf design stress

Test Equipment: W22 x 121 x 17 ft load frame, 10 ft deep Geopiers as uplift reactions, steel pedestal

(reaction system, reference beams & supports, gauges, jack, ram, calibrations, include all relevant info.): 100 ton jack w/ hand pump, hoses, load holding valve and 10,000 psi pressure gauge, serial no. 12345, calibrated 4/3/98
(3) 0.001 in. dial gauges, mounted to jack, w/ plungers set on aluminum channels anchored minimum 2 dia. from test Geopier
tightened strings of wire and graduated scales mounted on each end of load frame - used to check uplift reaction deflection

Load No.	Planned Stress (psf)	% Des. Stress	Ram Load (tons)	Gauge Pressure (psi)		Applied Stress (psf)	% Des. Stress	Time (hrs:min)	Dial Readings (in.)				Avg. Deflection (in.)	Uplift Deflection (in.)		Geopier Modulus (pci)	Remarks
				Planne	Actual				#1	#2	#3	Avg.		#1	#2		
0	500	5.0%	1.23	119	100	420	4.2%	8:00	0.237	0.278	0.313	0.276		0	0		seat load
1	1700	17.0%	4.17	405	400	1680	16.8%	8:01	0.270	0.296	0.353	0.306	0.030				
								8:16	0.280	0.307	0.360	0.316	0.040			210	
								8:31	0.282	0.307	0.364	0.318	0.042				
2	3300	33.0%	8.10	786	800	3360	33.6%	8:34	0.358	0.364	0.383	0.368	0.092				
								8:49	0.362	0.370	0.389	0.374	0.098			205	
								9:04	0.364	0.371	0.392	0.376	0.100				
3	5000	50.0%	12.27	1190	1200	5040	50.4%	9:05	0.399	0.405	0.431	0.412	0.136				
								9:20	0.406	0.411	0.450	0.422	0.146			208	
								9:35	0.409	0.414	0.455	0.426	0.150				
								9:50	0.412	0.418	0.460	0.430	0.154				
4	6700	67.0%	16.44	1595	1600	6721	67.2%	9:52	0.445	0.475	0.489	0.470	0.194				
								10:07	0.454	0.495	0.510	0.486	0.210			202	
								10:22	0.460	0.500	0.516	0.492	0.216				
								10:37	0.461	0.501	0.517	0.493	0.217				

Figure 5.2.2 Example Modulus (Compression) Load Test Data

Load No.	Planned Stress (psf)	% Des. Stress	Ram Load (tons)	Gauge Pressure (psi) Planne	Gauge Pressure (psi) Actual	Applied Stress (psf)	% Des. Stress	Time (hrs:min)	Dial Readings (in.) #1	Dial Readings (in.) #2	Dial Readings (in.) #3	Dial Readings (in.) Avg.	Avg. Deflection (in.)	Uplift Deflection (in.) #1	Uplift Deflection (in.) #2	Geopier Modulus (pci)	Remarks
5	8300	83.0%	20.37	1976	2000	8401	84.0%	10:39	0.521	0.531	0.541	0.531	0.255				
								10:54	0.535	0.541	0.551	0.542	0.266				
								11:09	0.541	0.550	0.551	0.547	0.271			201	
								11:24	0.548	0.552	0.555	0.552	0.276				
								11:39	0.548	0.553	0.556	0.552	0.276				
6	10000	100.0%	24.54	2381	2400	10081	100.8%	11:40	0.559	0.587	0.618	0.588	0.312				
								12:40	0.566	0.605	0.626	0.599	0.323				
								12:55	0.570	0.610	0.633	0.604	0.328			201	
								13:10	0.574	0.614	0.635	0.608	0.332				
								13:25	0.576	0.615	0.636	0.609	0.333				
7	11700	117.0%	28.72	2785	2800	11761	117.6%	13:27	0.601	0.634	0.661	0.632	0.356				
								14:27	0.611	0.644	0.671	0.642	0.366				
								14:42	0.627	0.657	0.686	0.657	0.381			204	
								14:57	0.633	0.661	0.690	0.661	0.385				
								15:12	0.635	0.663	0.691	0.663	0.387				
8	13300	133.0%	32.64	3166	3150	13231	132.3%	15:15	0.652	0.691	0.725	0.689	0.413				
								15:30	0.676	0.690	0.740	0.702	0.426				
								15:45	0.682	0.695	0.747	0.708	0.432			203	
								16:00	0.686	0.700	0.749	0.712	0.436				
								16:15	0.688	0.701	0.752	0.714	0.438				
9	15000	150.0%	36.82	3571	3550	14911	149.1%	16:17	0.712	0.752	0.798	0.754	0.478				
10	10000	100.0%	24.54	2381	2400	10081	100.8%	16:19	0.668	0.685	0.735	0.696	0.420				
																N/A	
11	6600	66.0%	16.20	1571	1550	6511	65.1%	16:21	0.572	0.575	0.721	0.623	0.347				
12	3300	33.0%	8.10	786	800	3360	33.6%	16:23	0.502	0.498	0.529	0.510	0.234				
13	0	0.0%						16:25									

Figure 5.2.2 (cont'd) Example Modulus (Compression) Load Test Data

geopier ™

Test Geopier Installation Record

Project: Geopier Foundation Manual Example

Design Load: 10000 Installation Date: 4/16/98 Weather: P.Cloudy, 60s

Installation Equip.: Samsung 210SE hydraulic excavator with
Tramac 900 hydraulic hammer and 26" tamper

Materials/ Suppliers: aggregate: washed #57 and crushed base course stone from local DOT certified quarry
uplift anchors: 1" dia. A36 rod w A325 bolts, 10"x28"x1" A36 bottom plate

Depth	Soil Description	As-Built Geopier, Dia. 30", Type: compression Stone	Casing	Remarks
0.0	Very loose, tan and gray silty SAND, wet below 3ft -- fill --		CASING NOT REQUIRED	1" thick steel bearing plate 18" below grade
		Graded Aggregate		DPT: 15-23
		Graded Aggregate		
		Graded Aggregate		slow water seepage
5.0	Medium dense silty SAND, trace clay and gravel, wet	Graded Aggregate		
		Graded Aggregate		
		#57		
		#57		BST: 5/8" after 15 seconds, 2" water @ bottom of hole
	Bottom of hole @ 8.5'			

Figure 5.2.2 Example Modulus (Compression) Load Test Data

Geopier Modulus Load Test
Deflection vs. Stress
Conducted for Geopier Reference Manual Example
on 4/20/98 near Borings B1, B4 and B7

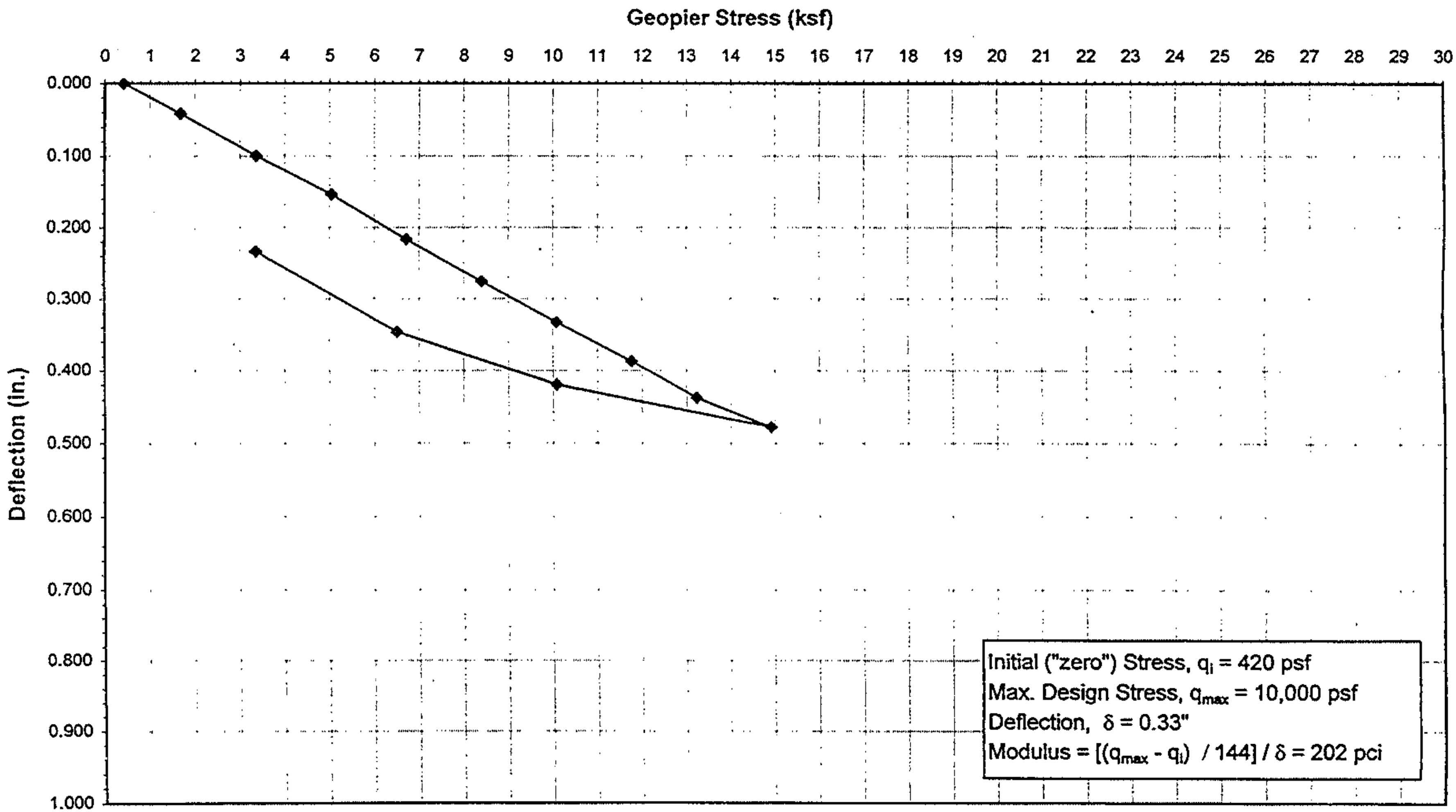


Figure 5.2.3 Example Geopier Stress vs. Deflection Curve

but rather to determine stiffness (modulus) of the pier at the design stress to be used for *settlement estimations*. There is, therefore, no hard and fast rule to extend the modulus load test to 150% of the Geopier element stress level, however, this is normally done in practice. Also, experience has shown that at stresses greater than about 150% of the design stress, the matrix soils and Lower Zone begin to contribute excessively to the calculation of the modulus of the Geopier element. This effect may also occur at loads less than 150%, but the effect is less pronounced.

5.2.2 Uplift Load Testing of Geopier Elements

The procedure for uplift load testing is based on portions of ASTM D 3689. Load increments are calculated using the same schedule as in Table 5.2.1, except that the maximum uplift load is used directly, with no need to convert from stress. Load durations, holding criteria, and documentation are the same as for the modulus (compression) test, unless special procedures are used to more accurately simulate a rapid uplift loading, such as for seismic loading simulations. The deflection recorded at the design load is then compared to acceptable limits established.

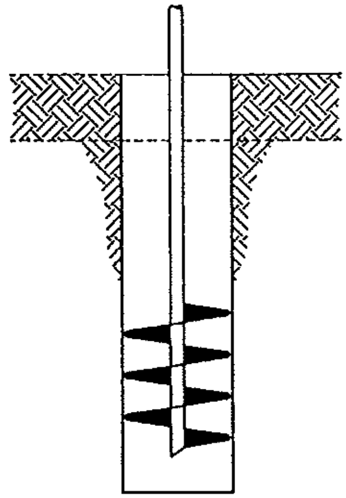
Often it is possible to conduct an uplift load test at the same time as the modulus load test, since uplift Geopier elements are generally used as anchor reactions for the modulus test load frame, as shown in Figure 5.2.1.

5.3 Geopier Foundation Construction Process

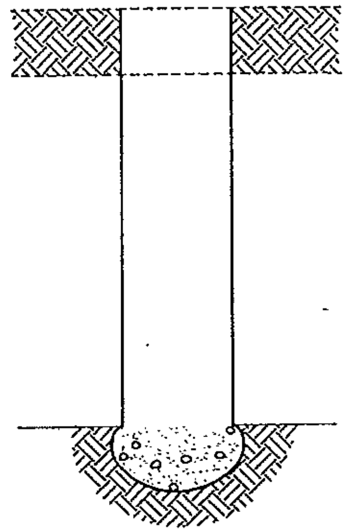
Installing Geopier elements is a relatively simple application of soil construction, and the tools used consist of a drill; a modified hydraulic hammer; and a beveled, circular-headed Geopier tamper specialized for the application. In Figure 5.3.1, the four-step process presents what happens during the Geopier foundation construction process. When difficulties arise during construction, whether obstructions are encountered while drilling, groundwater causes caving, or confusion exists regarding footing or site elevations, it is important to maintain open communications between all parties involved. The following paragraphs offer some insight into controlling the construction process. However, as with all forms of construction, when normal procedures cannot be applied, reliance on sound engineering judgment is necessary.

5.3.1 Quality Assurance (QA)

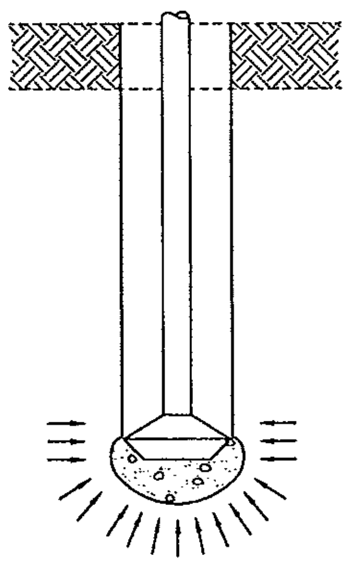
For Geopier foundation installation, QA services should normally be performed by the Geotechnical Engineer who performed the subsurface investigation. These services include monitoring installation of modulus load test piers, monitoring load tests, and monitoring daily pier installation, including observing subsurface conditions and soils during installation. The owner's QA representative often coordinates his daily activities with the Geopier foundation installer's full-time QC representative. In addition, the QA representative should monitor footing placement after Geopier elements have been installed to verify that footing bottoms are prepared properly, as should be done with any building foundation system.



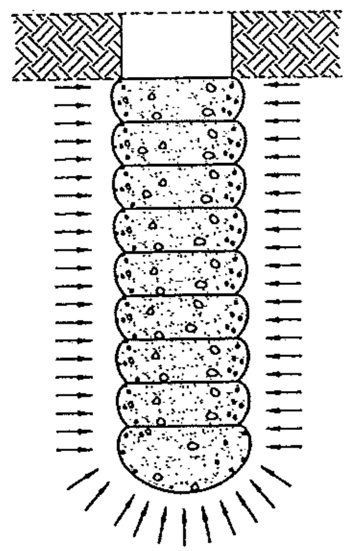
1. Make cavity



2. Place stone at bottom of cavity.



3. Make a bottom bulb. Densify and vertically prestress matrix soils beneath the bottom bulb.



4. Make undulated-sided Geopier shaft with 12-inch (or less) thick lifts. Build up lateral soil pressures in matrix soil during shaft construction. Use well-graded base course stone in Geopier element shaft above groundwater levels.

Figure 5.3.1 4-Step Process

5.3.2 Quality Control (QC)

The internal Geopier foundation installer's internal QC program is designed to assure verify that each Geopier is installed correctly. This is achieved by:

- coordinating footing layout, elevation and grading with the General Contractor
- observing soils encountered during drilling
- measuring drill depths and top elevations of Geopier elements
- controlling moisture content of aggregate within acceptable limits
- controlling and recording type and number of lifts of aggregate
- performing qualitative tests on production Geopier elements as appropriate
- implementing corrective measures when necessary, with approval of the Geopier foundation designer
- reporting construction activities to the owner's representative

Although the basics of the program are easily defined, the details depend on the specific project and site conditions. The experience of local construction and design professionals, as well as geology, may affect the QC requirements for a project. An example Geopier Foundation QC Report is shown in Figure 5.3.2.

5.3.3 QA/QC (QAC) Testing

As with other tried and true QAC testing programs in earthwork construction, Geopier QAC tests are indicators of the characteristics of the Geopier elements relative to each other and to the modulus load test pier. The goal is to determine a pattern of construction that produces good quality Geopier foundations at a given project site, and identifies conditions that may effect Geopier foundation quality.

5.3.3.1 Bottom Stabilization Tests (BSTA)

Bottom stabilization tests are a method of verifying that the Geopier element being installed has achieved a general stabilization prior to the completion of the installation. It is also a method to determine that production Geopier elements are comparable in quality to load test Geopier elements. This test may be performed on top of the bottom bulb, or after one or several lifts have been constructed on top of the bottom bulb. When the compacted aggregate and matrix soil becomes stiff enough to resist downward movement of the tamper, BSTA has been achieved. A pattern of successful BSTA observations is sufficient to reduce BSTA verification to spot checks. The specific procedure for verifying BSTA is as follows (reference Figure 5.3.3).

a) Apply tamper energy to the bottom lift of aggregate and compact aggregate for the same duration and number of passes as in the load test.
b) Turn off the energy source, place a reference bar over the cavity for the Geopier element and mark the tamper shaft at the reference bar.
c) Restart the energy for 15 seconds.
d) Stop the energy and mark the tamper shaft again at the reference bar.

geopier TM

Geopier Daily Q/C Summary

Project Name & Location: Geopier Foundation Manual Example

Equipment Used: Excavator and modified hydraulic hammer with 26" tamper

Geopier Project Number: 1212

Owner: XYZ Development

General Contractor: PDQ Construction

Date: 4/20/98

Geopier Installer: Geopier Foundation Company, Inc.

Time on Site: 7:00 am to 5:30 pm

Weather: Sunny, 70°

Sheet: 1 of 3

Geopiers Installed

Location	Geopier No.	Bottom of Footing EL., ft (1)	Geopier Shaft Length, ft (2)	Ground Surface EL., ft (3)	Geopier Bottom EL., ft (4)=(1)-(2)	Geopier Drill Depth, ft (3)-(4) Planned	Geopier Drill Depth, ft (3)-(4) Actual	Geopier Top EL., ft (5)=(1)+0.5	Geopier Top Depth, ft (3)-(5) Planned	Geopier Top Depth, ft (3)-(5) Actual	Geopier Stone Lifts #1	Geopier Stone Lifts #2	Remarks
C-3	A	175.00	7.00	176.00	168.00	8.00	8.25	175.50	0.50	0.33	1	6	BST: 3/8", DPT: 10-23
	B	175.00	7.00	176.00	168.00	8.00	8.10	175.50	0.50	0.00	2	6	6 inches water at bottom of hole
	C	175.00	7.00	176.00	168.00	8.00	8.33	175.50	0.50	0.00	1	6	
	D	175.00	7.00	176.00	168.00	8.00	8.00	175.50	0.50	0.25	1	7	
C-4	A	174.00	9.00	176.00	165.00	11.00	11.50	174.50	1.50	1.33	2	9	4 inches water at bottom of hole
	B	174.00	9.00	176.00	165.00	11.00	11.00	174.50	1.50	1.25	1	9	
	C	174.00	9.00	176.00	165.00	11.00	10.83	174.50	1.50	1.00	1	8	
	D	174.00	9.00	176.00	165.00	11.00	11.17	174.50	1.50	1.50	1	9	
	E	174.00	9.00	176.00	165.00	11.00	11.25	174.50	1.50	1.50	1	10	DPT: 12-24
	F	174.00	9.00	176.00	165.00	11.00	11.00	174.50	1.50	1.33	1	8	
B-4	A	174.00	9.00	176.00	165.00	11.00	11.50	174.50	1.50	1.33	2	9	3 inches water at bottom of hole
	B	174.00	9.00	176.00	165.00	11.00	10.83	174.50	1.50	1.00	1	8	
	C	174.00	9.00	176.00	165.00	11.00	10.83	174.50	1.50	1.00	1	8	
	D	174.00	9.00	176.00	165.00	11.00	11.17	174.50	1.50	1.50	1	9	
	E	174.00	9.00	176.00	165.00	11.00	10.83	174.50	1.50	1.00	1	8	
	F	174.00	9.00	176.00	165.00	11.00	11.17	174.50	1.50	1.50	1	9	
D-3	A	174.00	9.00	176.00	165.00	11.00	11.17	174.50	1.50	1.50	1	9	DPT: 9-23
	B	175.00	7.00	176.00	168.00	8.00	8.10	175.50	0.50	0.00	2	6	6 inches water at bottom of hole
	C	175.00	7.00	176.00	168.00	8.00	8.33	175.50	0.50	0.00	1	6	
	D	175.00	7.00	176.00	168.00	8.00	8.00	175.50	0.50	0.25	1	7	

General Comments: Installed 35 Geopiers in general accordance with design and load test requirements. Graded aggregate was wetted several times during the day's production to maintain proper moisture content to achieve densification. No Geopiers were installed that did not meet project requirements and Geopier specifications. Geopiers were located in the field by the General Contractor's Surveyor.

G. O. Pier
Geopier Installer's Q/C Representative

R. U. Watching
Owner's Q/A Representative

Figure 5.3.2 Example Quality Control Data

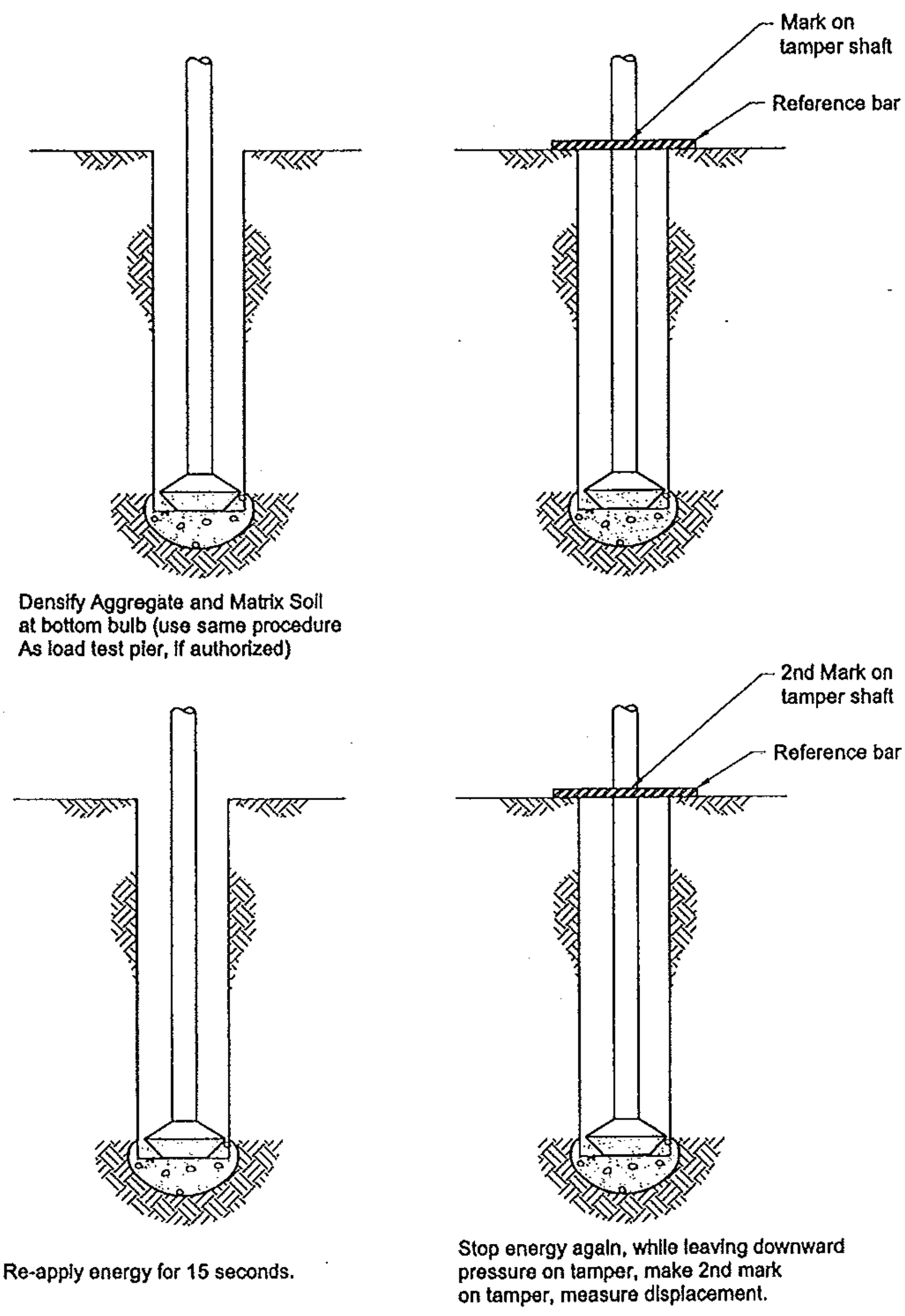


Figure 5.3.3. Schematic Bottom Stabilization Test

e) The downward movement of the tamper shaft is the distance between the two marks. If displacement is less than 1.5 times the value observed during construction of the modulus load test pier(s), BSTA has been achieved.
f) If BSTA has not been achieved, continue normal construction and repeat steps a-e on successive aggregate lifts.
g) If BSTA is not achieved in the lower 2/3 of the Geopier element, stop construction, verify the aggregate compaction as described below, redrill the Geopier cavity and rebuild the Geopier, unless directed otherwise by the Geopier designer.

BSTA is generally related to the stiffness of the matrix soils and their potential to dissipate pore pressures, the size of the open graded aggregate used at the bottom of the Geopier cavity, the duration and number of tamping passes, and the size of the tamper foot relative to the energy of the hydraulic hammer. *When tamping energy is excessive in relation to the strength of underlying and adjacent matrix soils, BSTA testing is not appropriate.*

5.3.3.2 Dynamic Cone Penetrometer (DPT) Testing

The DPT is used in general accordance with ASTM STP 399 to verify graded base course aggregate densification within the top few feet of the Geopier element after tamping energy has been applied. The DPT need not be used on every Geopier element, or on a continuous basis unless unacceptable DPT results, questionable aggregate moisture content or questionable aggregate gradation, require additional DPT verification of aggregate densification. If average penetration resistance measured consistently exceeds 15 blows, and less than 10% of tests fall below 15 blows per 1.75 inches, then testing may be reduced to spot checks. If average penetration resistance falls below 15 blows, additional tamping energy, improved aggregate moisture control or different aggregate may be required. Dynamic penetration testing is inappropriate for clean stone and is only used for graded base course stone.

5.3.3.3 Detecting Negative Pore Pressure Build-Up in Soft Clays

On rare occasions, fine-grained soils will develop negative pore water pressures. When suspected to exist, this condition can be identified in the field by using a *modified bottom stabilization test*. After performing the normal test, wait 15 minutes and repeat the test. If deflection is greater after the delay, this indicates that the soil has negative pore water pressures. Use of an open-graded stone for the lower bulb of the Geopier element reduces the severity of this condition.

5.4 Construction Considerations

There are several aspects of the installation process that effect the performance of Geopier elements which are not performed by the Geopier foundation installer. These items are usually the responsibility of the General Contractor and are discussed in the following sections.

5.4.1 Protection of Geopier Elements

Before, during and after installation, consideration should be given to the overall construction schedule and how that may affect Geopier foundations. Trenching operations for site and building utilities can disturb Geopier foundations if they are too close, and should be coordinated with the Geopier foundation designer. As with all earthwork projects, managing surface and subsurface water is important to maintain a controlled building pad. Geopier elements are usually topped off several feet below working grade, in which case the open holes are typically filled to the ground surface with loosely placed drilling spoils by the Geopier foundation installer, or in some cases they may be protected with a thin concrete cap of equal diameter.

5.4.2 Footing Excavation and Placement

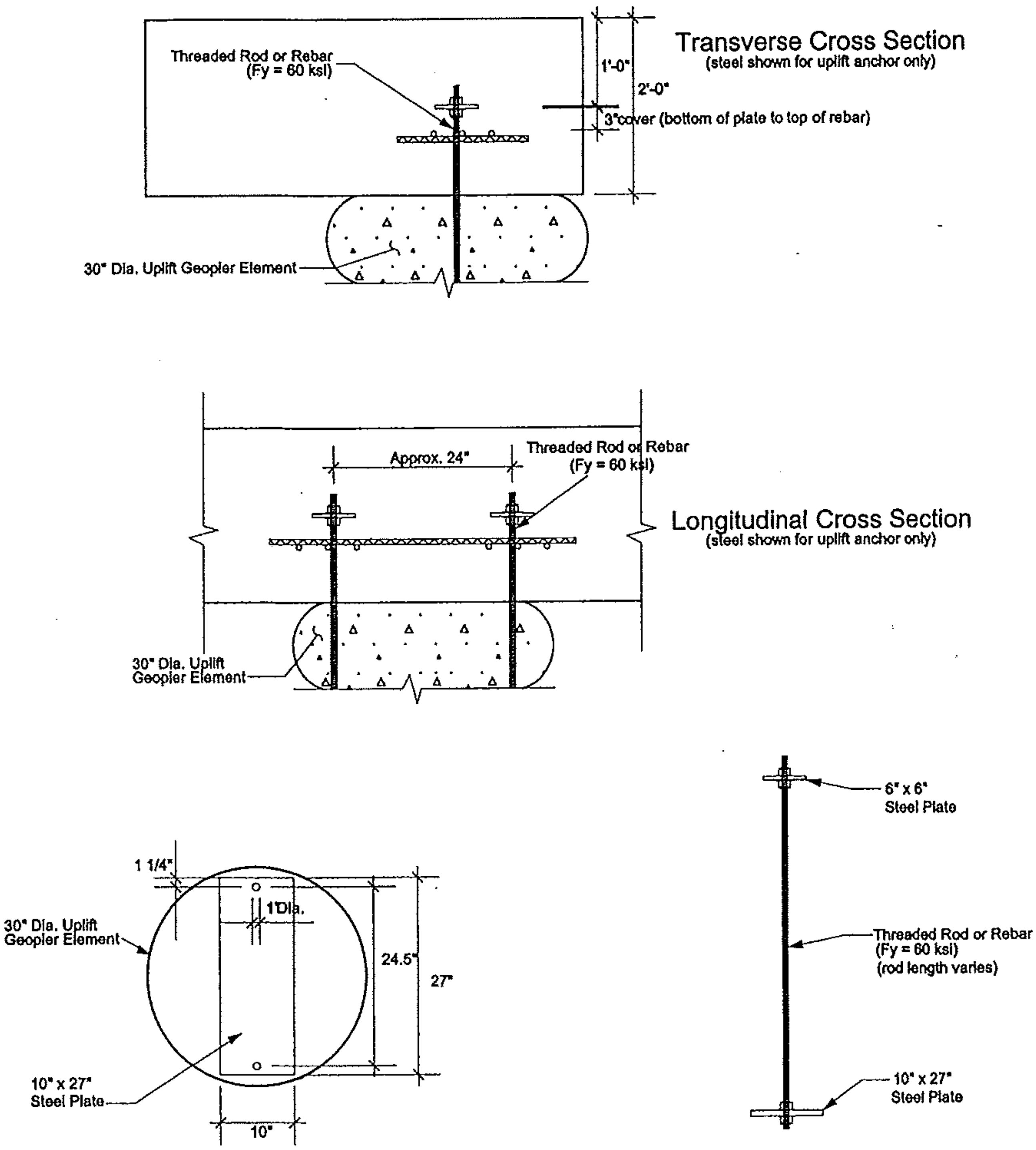
There is no difference between excavation and placement of concrete for Geopier-supported footings with that for conventional spread footings. Each case requires inspection of the footing subgrade. Is the bottom of the footing excavation free from standing water, loose material, and other deleterious material? One concern for Geopier supported foundations is that excessively deep excavations can be more difficult to correct with Geopier elements than conventional spread footings. The main requirements for preparing Geopier-supported footings are limiting overexcavation to three inches (including that caused by digging teeth), and tamping the bottom of the footing excavation and tops of exposed Geopier elements with a "jumping jack" type of hand-held mechanical compactor prior to placing concrete.

5.4.3 Soil Reinforcement Applications

Special applications such as slope or embankment stabilization and retaining wall support with Geopier reinforcements may not have the same design requirements as Geopier-supported foundations. For example, Geopier elements used for slope stability do not require footing preparation procedures. Retaining walls may be designed using Geopier foundations to resist sliding loads, in which case particular care must be taken to assure that intimate contact between the retaining wall footing and the Geopier top is maintained. Yet another application is combining the settlement control and global stabilization function for embankments, or applying the same to mechanically stabilized earth (MSE) structures, such as segmental retaining walls. In these MSE applications, there is no stiff structural element to transfer stresses to the Geopier foundation. A geotextile or geogrid mattress using aggregate backfill is often employed to provide more efficient stress concentration on Geopier foundations for such applications.

5.4.4 Uplift Anchors

Details for uplift anchor connections (Figure 5.4.1) are usually coordinated with the Structural Engineer. It is important to utilize the same hardware in production Geopier elements as the load test Geopier element(s).



Plan View of Plate at Bottom of Geopier Element

Uplift Anchor Assembly Cross Section

Figure 5.4.1. Example Uplift Anchor Detail

5.5 Special Installation Considerations

Construction of Geopier elements takes place in a wide variety of conditions, some favorable and some unfavorable. As with other earthwork construction, water (surface and subsurface) and unstable soils often present the greatest challenges. Over the past several years the various licensed Geopier foundation installers have developed methods to deal with these challenges, as described in the following subsections.

5.5.1 Construction Below the Water Table

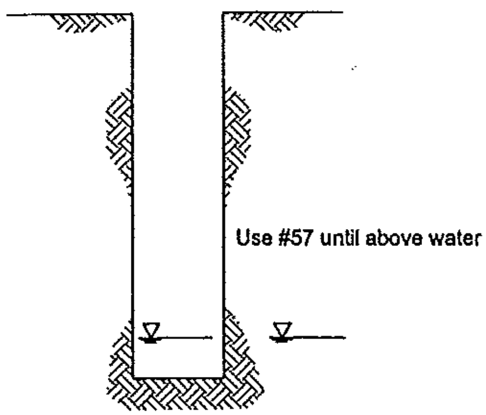
Since Geopier elements are installed relatively quickly (typically 10 to 30 minutes per element), there is little time for groundwater to infiltrate into and fill up the cavity of a Geopier element, except when high permeability matrix soils are being reinforced. If coordination of drilling and tamping operations is handled properly, and cavities are not left open for very long, a high water table or perched water usually has little effect on Geopier construction. Except in soils with relatively high hydraulic conductivity (such as with clean sands and gravels), standard Geopier foundation installation procedures normally provide sufficient time to build in the dry or with limited trapped water, and no special materials or methods are required (Figure 5.5.1, case 1 and 2).

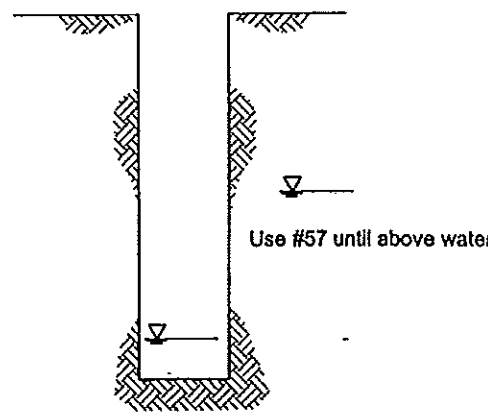
If the accumulated depth of water at the bottom of the cavity stabilizes at only a one or two foot height above drilled bottom depth, aggregate selection is the most effective means of addressing the problem. Clean, open-graded aggregate, such as #57 stone, can be placed and tamped in relatively shallow water. Successive lifts of open-graded aggregate are used until there is less than an inch or two of water standing on top of the preceding, compacted lift of aggregate. After that the Geopier element can be completed using graded aggregate base course stone and standard procedures (Figure 5.5.1, case 3 and 4).

Groundwater becomes a significant challenge when it accumulates rapidly in the cavity and to depths greater than several feet. If this is the case, caving might be a greater challenge than the water alone. Some means of groundwater pumping may be required to control deep water in the Geopier cavity. Pumping may be performed either directly in the Geopier element being constructed or by establishing sump pits or other methods to draw down the groundwater table in the general vicinity of the Geopier element construction. The selection of which method to use depends on site conditions and on the Geopier foundation installer's or General Contractor's preference. Casing may be required when excessive groundwater is present. Use of casing with Geopier foundation construction is discussed in detail in Section 5.5.3.

5.5.2 Wet Weather

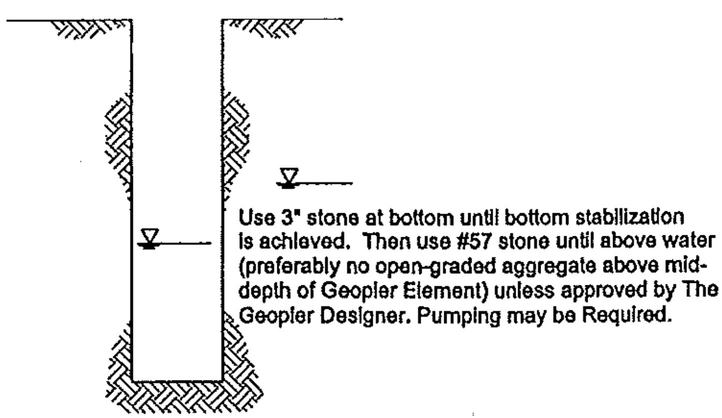
The main concern with wet weather is aggregate moisture control. Occasional heavy showers or thunderstorms can be dealt with by protecting aggregate stockpiles with plastic sheeting. For extended periods of heavy rain, the source aggregate arriving at the site may already be excessively wet of optimum moisture content. In such cases it

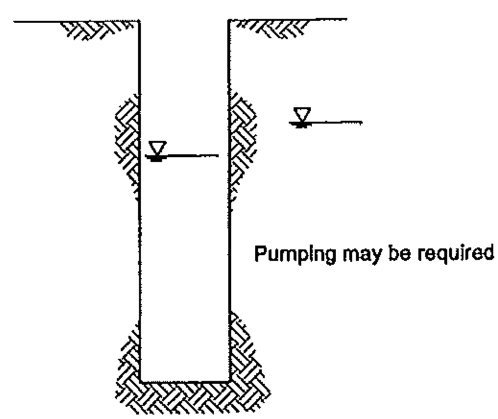




a) Groundwater within one foot of bottom of Geopier Element.

b) High groundwater with slow infiltration. Accumulates up to one foot before installation begins.





c) High groundwater table with rapid accumulation, but no caving.

High groundwater table with rapid accumulation, no caving and water depth above mid-depth of Geopier Element.

NOTE: Open graded aggregate shall be used where standing water is > 1" deep, including standing water above a compacted lift of aggregate. Crushed base course stone shall not be placed if standing water is > 1" deep.

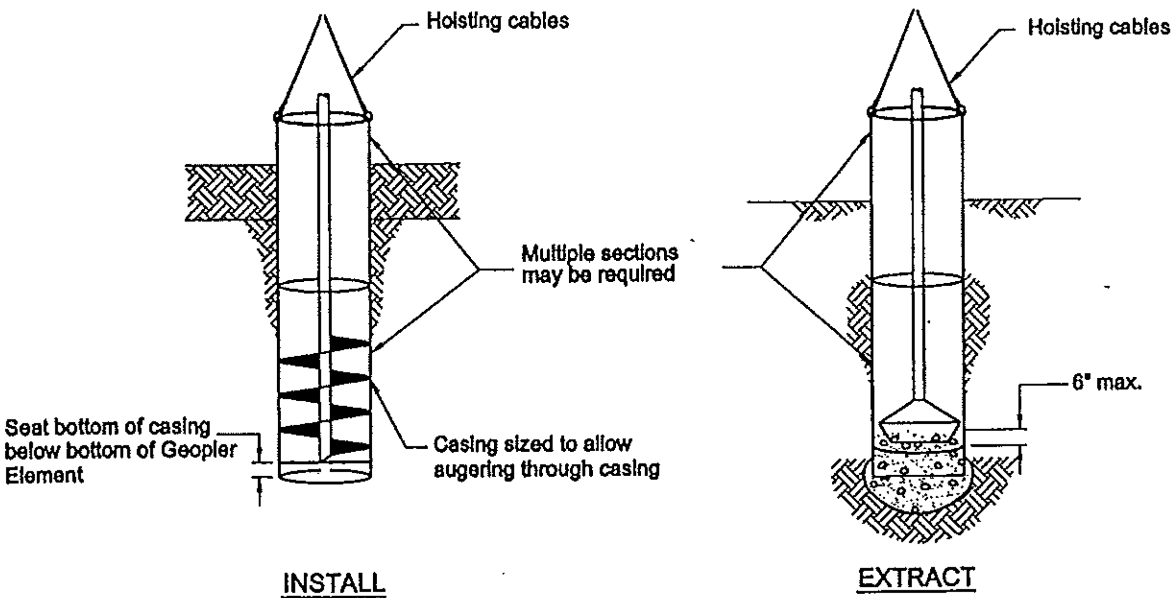
Figure 5.5.1. Construction Below the Groundwater Table

may be desirable to use graded base course aggregate that has a lower percentage of fine soil grains. Without the presence fine soil grains, on-site stockpiles can drain more freely, and tamping will not result in excessive pore pressure buildup in moist aggregate. With fines present in the graded base course aggregate, pore pressure buildup prevents proper compaction of wet aggregate.

5.5.3 Caving Soils

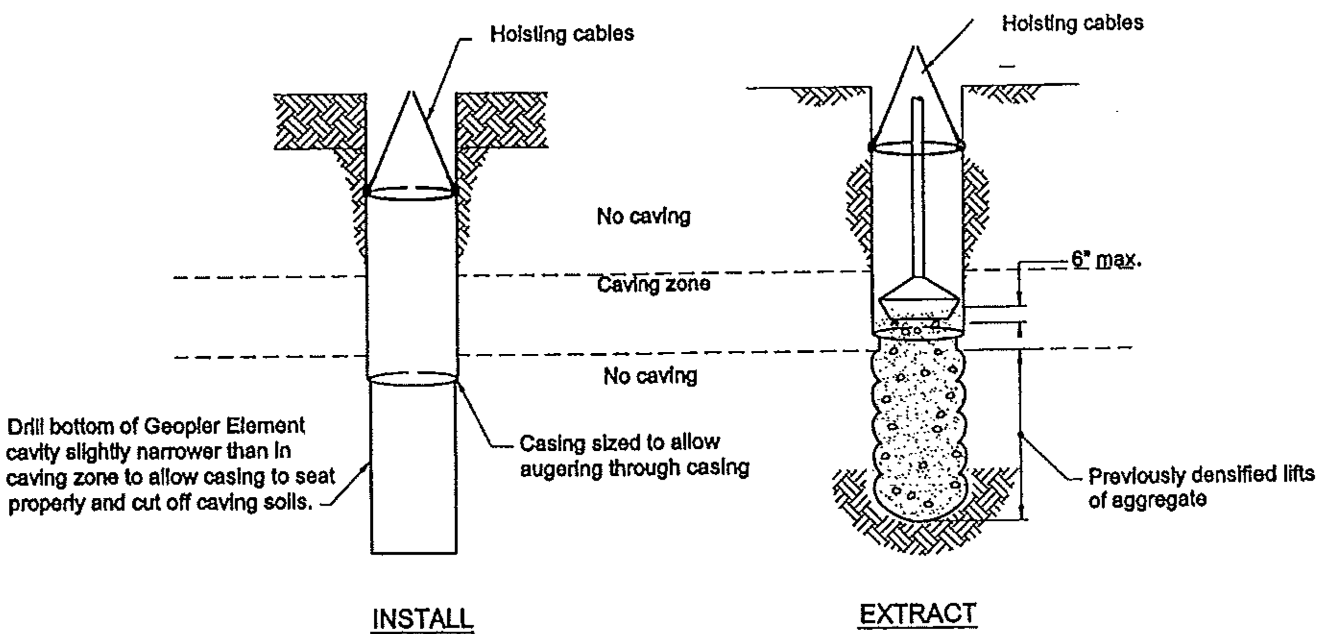
How caving soils are handled depends on the degree to which caving occurs. Minor amounts of caving (less than about 10% of the volume of each lift of aggregate) can be tolerated, and often occurs. In these situations, the small volume of matrix soil that caves into the Geopier element does not prevent good compaction of the Geopier aggregate, and cleaning out the caving spoils is not required. However, if significant caving occurs on a lift of aggregate, removing and replacing the contaminated aggregate may be required. When caving materials are relatively clean sands, they may be compacted and utilized as a portion of the Geopier element. The worst caving situation occurs where soils have little or no cohesion and cannot support open cavities.

When caving soils prevent the drilling of cavities for Geopier elements altogether, casing is employed to stabilize the cavities (Figure 5.5.3). Aside from the additional time and materials required there are two main casing considerations - advancing and extracting the casing. When advancing the casing, it is sometimes necessary to apply downward force to push the casing into place. Although the casing can often settle under its own weight, it may be desirable to push the casing such that the bottom of the casing is located a few inches below the bottom of the drilled cavity. This is done to reduce the potential for quick conditions (in the presence of high groundwater) and also to prevent the development of an excessively large annular space between the casing and the sides of the drilled cavity. For layered soils where cohesive soils underlie the caving soils, it may be necessary to push the casing into the cohesive soils to "seal off" the caving zone. In such instances, full depth casing may not be required. When extracting the casing, it is necessary to first place a loose lift of aggregate, and then slowly pull the casing until its bottom is just below the top of the loose aggregate (within 2 to 4 inches of the top of the loose aggregate). Next, tamp the loose aggregate, add another lift, and repeat the process until the casing is removed from the caving soils. It is important that the tamping energy be transmitted below the bottom of the casing, otherwise the lateral stress buildup in the matrix soils is not effectively achieved. In addition, a plug of compacted aggregate is created at the bottom of the casing, which may be difficult to extract from the casing.



a) Caving soils present for full depth of Geopier Element.

NOTE: Place loose aggregate and extract casing such that bottom of casing is within 6 inches of top of loose lift of aggregate. Compact aggregate as with non-cased Geopier Elements. Repeat aggregate placement, casing extraction, and tamping on successive lifts.



b) Caving soils confined by layers of stable soils.

Figure 5.5.3. Casing Geopier Elements in Caving Soils

6.0 FUTURE OUTLOOK FOR GEOPIER FOUNDATIONS

We expect the Geopier Foundation System to continue its controlled growth in the U.S. for the foreseeable future. The technology is a proven innovation that refines and improves on the centuries-old methods of removal and replacement. Use of the system is expanding as more engineers, architects, builders and owners become familiar with the advantages and characteristics of soil reinforcement using the Geopier technology.

Factors that will influence the future growth of Geopier foundation systems throughout the United States include:

1. Wider recognition of Geopier foundation systems as a valid and practicable solution for settlement control of buildings and other structures.

2. Continued expansion of applications for Geopier soil reinforcements beyond providing settlement control. Areas for expansion include uplift anchors, lateral load resistance control, applications in special problem areas such as seismic zones, highly expansive soils, frozen soils, global stabilization for landslide control, as well as retaining wall and embankment slope stabilization.

3. Continued improvement in the understanding of the load transfer mechanisms within Geopier elements and how Geopier foundations work. Current research and in-situ testing in the areas of stress distribution, contribution of lateral stress increase to settlement control, and uplift control mechanisms, will improve our understanding and will result in better, less conservative design methods.

4. Continued improvement in apparatus and installation techniques that will increase production rates and help lower Geopier foundation construction costs to maintain competitiveness in the marketplace.

5. Improved design methods and tools to provide guidance to geotechnical engineering firms to assist them in understanding, recommending, and monitoring the design of Geopier foundation systems.

6. Improvements in Quality Control and Quality Assurance methods.

The goal of Geopier technology is to become and remain the leader in vertical soil reinforcement and intermediate foundation design and construction throughout the United States.

7.0 REFERENCES

Barksdale, R. D., and R. C. Bachus, (1983). "Design and Construction of Stone Columns, Volume 1," *U.S. Department of Transportation, Federal Highway Administration Report No. FHWA/RD-83/026.*

Bowles, Joseph, E., (1988). *Foundation Analysis and Design*, McGraw Hill, 3rd Edition, pp. 452-460.

Bowles, Joseph, E., (1977). *Foundation Analysis and Design*, McGraw Hill, 2nd Edition, p. 35.

Burmister, D.M., (1952). "Stress and Displacement Characteristics of a Two-Layered Rigid Base Soil System: Influence Diagrams and Practical Applications," *Proceeding of the Highway Research Board,* 35, pp. 773-814.

Fox, E.N. (1948). *The Mean Elastic Settlement of a Uniformly-Loaded Area at a Depth Below the Ground Surface*, 2nd ICSMFE, Vol. 1, pp. 129-132.

Handy, Richard L., et al., (1982). "In-Situ Stress Determination by Iowa Stepped Blade." ASCE J GED 108, GT11, pp. 1405-1422.

Handy, Richard L., (1997). Personal Correspondence on Stepped Blade Testing Adjacent to Geopiers Installed in Clay Soils.

Lambe, T. W., and R. V. Whitman, (1969). *Soil Mechanics,* John Wiley and Sons, pp. 146-149.

Lawton, E. C., N. S. Fox, and R. L. Handy, (1994). "Control of Settlement and Uplift of Structures Using Short Aggregate Piers," *Proceeding of In-Situ Deep Soil Improvement Conference,* ASCE National Convention, October, 1994, pp. 121-132.

Lawton, E. C., and N. S. Fox, (1994). "Settlement of Structures Supported on Marginal or Inadequate Soils Stiffened with Short Aggregate Piers," *ASCE Special Technical Publication, No. 40,* pp. 962-974.

Lawton, E. C., (1995). "Soil Stabilization and Improvement: Non-Grouting Techniques," Section 5A in *Practical Foundation Engineering Handbook,* McGraw-Hill, R. W. Brown, editor, pp. 5.1-5.276.

Martin, R. E., (1987). "Settlement of Residual Soils," *Proceedings of ASCE, Foundations and Excavation in Decomposed Rock of the Piedmont Province, ASCE Geotechnical Special Publication No. 9, pp. 1-14.*

Mitchell, J. K., (1993). *Fundamentals of Soil Behavior,* 2nd edition, New York, John Wiley & Sons.

Mitchell, J. K., and W.S. Gardner, (1975). "In-Situ Measurement of Volume Change Characteristics," 6th PSC, ASCE, vol. 2, pp. 279-345.

Poulous, H.G., and E. H. Davis, (1974). *Elastic Solutions for Soil and Rock Mechanics,* John Wiley & Sons, Inc., New York.

Schmertmann, (1975). "The Measurement of In-Situ Shear Strength," 6th PSC, ASCE, vol. 2, pp. 57-138.

Skempton, A. W., (1944). "Notes on the Compressibility of Clays," *Quarterly Journal of the Geological Society,* London, C, pp. 119-135.

Spangler, M.G., and R.L. Handy, (1982). *Soil Engineering,* Harper & Row, 4th edition, New York.

Terzaghi, K., and Peck, R. B., (1967), "Soil Mechanics in Engineering Practice," 2nd Edition, John Wiley and Sons, New York, 729 pp.

Appendix A
Geopier Foundation System Specifications

- General Recommendations for a Geotechnical Report
- Performance Specifications for Design-Build Projects

Example Recommendation for Geopier Foundations

Geopier Foundations may be used to reinforce the subsoils on this site to support relatively high capacity shallow footings (mats, slabs). The Geopier foundation system, which has been in use since 1988, is a practical refinement on the traditional over-excavation and replacement method of strengthening subsoils for settlement control and bearing capacity improvement. The Geopier support elements are constructed by drilling a hole to create a cavity, removing a volume of compressible subsoil materials, then building a bottom bulb of clean, open-graded stone while vertically prestressing and prestraining subsoils underlying the bottom bulb. The Geopier shaft is built on top of the bottom bulb, using well-graded highway base course stone placed in thin lifts (12 inches compacted thickness) above groundwater levels. For shaft portions that may exist within water, clean aggregate is used. Densification of the bottom bulb and of the undulating shaft lifts is accomplished by using the impact ramming action of a modified hydraulic hammer. The tamper consists of a special steel alloy shaft and a round, beveled tamper head. The beveled tamper head assists in transferring force laterally during impact densification, resulting in pushing of aggregate against the confined walls of the cavity. The nature of soil is to "push back," creating significant lateral soil pressure build-up in the matrix soil and lateral confinement to the Geopier elements. In addition to increasing shear resistance at the Geopier element perimeter, the increased horizontal stress in the matrix soil improves the matrix soil and makes it stiffer.

For this project, Geopier elements with shaft lengths of ___ to _____ **feet** can be expected to provide a support capacity of ___ **kips** for each 30-inch diameter pier and its representative footing (mat) segment. Footings (mat) can be designed for an allowable composite bearing pressure of _________ **psf.**

The licensed Geopier foundation installer, ________________________________, Inc., would design and install Geopier foundations for this project. Geopier Foundation Company, Inc. will provide an internal peer review and approval of design. Responsibility for settlement performance is accepted by this Geopier design-build team. We recommend that you contact ____________________________to review and analyze the subsurface data contained in this report using available structural load and design information. After designing the support system, they will provide a cost and time estimate. The estimate should include the cost to provide a full-scale Geopier Modulus Load Test on site to verify design assumptions. The test provides a conservative measure of the stiffness of the Geopier element and will help establish installation procedures for this particular project. We can coordinate with the Geopier installer to locate the Modulus Load Test in the weakest site area and to provide full-time Quality Assurance monitoring services during Modulus Load Test operations.

We recommend that the Geopier installer's operations be monitored by us full-time as a Quality Assurance service. Our service will supplement the installer's internal QC program. Together the QA/QC program will monitor drill depths, Geopier element lengths, average lift thicknesses, installation procedures, aggregate quality, and densification of lifts. These items will be documented for each Geopier element installed to provide a complete installation report.

EXAMPLE SPECIFICATION FOR
SHORT AGGREGATE PIER FOUNDATION SYSTEMS

PART 1: GENERAL REQUIREMENTS

1.01 Description

Work shall consist of designing, furnishing and installing aggregate pier foundations to the lines and grades designated on the project foundation plan and as specified herein. The aggregate piers shall be constructed by compacting aggregate in an excavated hole using special high-energy impact densification equipment. The aggregate piers shall be in a columnar-type configuration and shall be used to produce an intermediate foundation system for support of foundation loads.

1.02 Approved Installers

A. Installers of aggregate pier foundation systems shall be licensed by Geopier Foundation Company, Inc. and shall have demonstrated experience in the construction of similar size and types of projects. They shall be approved by the Owner's Engineer. The Installer must be approved two weeks prior to bid opening. Installers currently approved for these works are:

1.03 Related Work

A. Section ____________ - Site Preparation
B. Section ____________ - Foundations
C. Section ____________ - Geotechnical Report and Recommendations

1.04 Reference Standards

A. Design

1. "Control of Settlement and Uplift of Structures Using Short Aggregate Piers," by Evert C. Lawton (Assoc. Prof., Dept. of Civil Eng., Univ. of Utah), Nathaniel S. Fox (President, Geopier Foundation Co., Inc.), and Richard L. Handy (Distinguished Prof. Emeritus, Iowa State Univ., Dept. of Civil Eng.), reprinted from *IN-SITU DEEP SOIL IMPROVEMENT, Proceedings of sessions sponsored by the Geotechnical Engineering Division/ASCE in conjunction with the ASCE National Convention held October 9-13, 1994, Atlanta, Georgia* .

2. "Settlement of Structures Supported on Marginal or Inadequate Soils Stiffened with Short Aggregate Piers," by Evert C. Lawton and Nathaniel

S. Fox. *Geotechnical Special Publication No. 40: Vertical and Horizontal Deformations of Foundations and Embankments,* ASCE, 2, 962-974.

B. Modulus Load Testing

1. ASTM D-1143 - Pile Load Test Procedures
2. ASTM D-1194 - Spread Footing Load Test
3. ASTM D-3687 - Uplift Load Test

C. Materials and Inspection

1. ASTM D-1241 - Aggregate Quality
2. ASTM STP 399 - Dynamic Penetrometer Testing
3. ASTM D-422 - Gradation of Soils

D. Where specifications and reference documents conflict, the Architect/ Engineer shall make the final determination of the applicable document.

1.05 Certifications and Submittals

A. The Aggregate Pier Installer shall submit detailed design calculations and construction drawings to the Owner or Owner's Engineer for approval at least 1 week prior to the start of construction. All plans shall be sealed by a Professional Engineer in the State in which the project shall be constructed.

B. The Aggregate Pier Installer shall submit a notarized manufacturer's certification prior to start of work, stating that the aggregate and other materials used meet the requirements of this specification.

C. Daily Aggregate Pier Progress Reports - The Testing Agency shall furnish a complete and accurate record of aggregate pier installation to the General Contractor. The record shall indicate the pier location, length, average lift thickness, and final elevations of the base and top of pier. The record shall also indicate the type and size of the densification equipment used. The Aggregate Pier Installer shall immediately report any unusual conditions encountered during installation to the General Contractor, to the aggregate pier designer, and to the Testing Agency.

1.06 Method of Measurement

A. Measurement of the aggregate piers is on a per-pier basis.

B. Payment shall cover design and installation of the aggregate pier foundation system. Excavation of unsuitable materials, drilling obstructions, delays, and remobilization as documented and approved by the Owner or Owner's Engineer, shall be paid for under separate pay items.

C. Quantities of piers, as shown on plans, may be increased or decreased at the direction of the Owner or Owner's Engineer, based on construction procedures and actual site conditions.

1.07 Basis of Payment

A. The accepted quantities of aggregate piers will be paid per approved, in-place aggregate pier. Payment will be made under:

Pay Item	Pay Unit
Aggregate Pier	Each

B. Unit prices shall be provided to account for:

Additional Installed Piers (w/o remobilization)	$__________ Each
Add for Casing Holes	$__________/LF
Additional Mobilizations	$__________ Each
Modulus Load Tests	$__________ Each

PART 2: PRODUCTS

2.01 Aggregate

Aggregate used for piers constructed above the water table shall be Type I Grade B in accordance with ASTM D-1241-68, or shall be other graded aggregate selected by the Aggregate Pier Installer and successfully used in the load test. It shall be compacted to a densification and strength which provides resistance to the dynamic penetration test (ASTM STP 399) of a minimum average of 15 blows per 1.75 inch vertical movement.

The number of tests performed during a workday by the Testing Agency shall depend on the consistency of achieving this minimum penetration resistance. Penetration tests need not be performed on every pier, nor on a continuous basis. If average penetration resistances measured exceed 15 blows, and less than 10% of tests fall below 15 blows, then testing may be reduced to spot checks. A pattern of successful tests is sufficient to reduce testing to several tests per day. Observation of questionable aggregate moisture content or questionable aggregate gradation appearance may determine the need for additional dynamic penetration testing to verify that proper densification and strength are being achieved.

For aggregate used for piers constructed below the water table, the gradation shall be the same as Type I Gradation B, except that particles passing the No. 40 sieve shall be eliminated. Alternately, No. 57 stone or other stone selected by the Aggregate Pier Installer may be used. Dynamic penetration resistance testing is inappropriate for this material.

PART 3: DESIGN REQUIREMENTS

The design submitted by the Aggregate Pier Installer shall consider the bearing capacity and settlement of all footings supported by aggregate piers, and shall be in accordance with acceptable engineering practice and these specifications. Total and differential settlement shall be considered. The design life of the structure shall be 50 years, unless specified by the Owner.

3.01 Aggregate Pier Design

Aggregate piers shall be designed in accordance with generally-accepted engineering practice and the method described in "Control of Settlement and Uplift of Structures Using Short Aggregate Piers," by Evert C. Lawton, Nathaniel S. Fox , and Richard L. Handy, reprinted from *IN-SITU DEEP SOIL IMPROVEMENT, Proceedings of sessions sponsored by the Geotechnical Engineering Division/ASCE in conjunction with the ASCE National Convention held October 9-13, 1994, Atlanta, Georgia.* The design shall meet the following criteria:

Maximum Allowable Bearing Pressure for Aggregate Pier Improved Soil	_______ psf
Minimum Aggregate Pier Area Coverage (Spread Footings)	30% *
Estimated Total Long-Term Settlement for Footings	$\leq$ 1 inch *
Estimated Long-Term Differential Settlement for Adjacent Footings	$\leq$ 0.5 inches *

* May change depending on designated projects requirements

3.02 Capacity and Size of the Aggregate Piers

The size and spacing of the aggregate piers are described on the foundation drawings. The Installer shall be responsible for delivering a system that will support the structure, while controlling settlement in accordance with these specifications. The Engineer shall approve any modifications in size and spacing of the aggregate piers, unless such modifications result in a more conservative design, in which case the Installer may approve them.

3.03 Design Submittal

The Aggregate Pier Installer shall submit 4 sets of detailed design calculations, construction drawings, and shop drawings for approval at least 2 weeks prior to the beginning of construction. A detailed explanation of the design properties for settlement calculations shall be submitted with the design. Additionally, the quality control test program for aggregate piers, meeting these design requirements, shall be submitted. All computer-generated calculations and drawings shall be prepared and sealed by a Professional Engineer, licensed in the State or Province where the piers are to be built.

PART 4: CONSTRUCTION

4.01 Quality Assurance/Quality Control

The Aggregate Pier Installer shall have a full-time Quality Control representative to verify and report all QC installation procedures. The Testing Agency shall provide Quality Assurance services and shall monitor the load tests when load tests are to be performed. The Testing Agency shall monitor the installation of load test aggregate piers to document procedures and criteria used for constructing the load test pier(s). The Aggregate Pier Installer shall provide and install all dial indicators and other measuring devices. The Testing Agency, paid for by the General Contractor or Owner, shall monitor the installation of aggregate piers. The Installer shall adhere to all methods, standards, and codes described herein, unless authorized in writing by the Engineer.

4.02 Layout of Aggregate Piers

The General Contractor shall provide layout of aggregate pier-supported footings, mats, or grade beams for this project, including layout of piers. The General Contractor shall provide ground elevations in sufficient detail to estimate drilling depth elevations to within 2 inches.

4.03 Excavation

Should any obstruction be encountered during drilling or excavation for aggregate piers, the General Contractor shall be responsible for removing such obstruction, or the pier shall be relocated or abandoned. Obstructions include, but are not limited to, boulders, timbers, concrete, bricks, utility lines, etc., which shall prevent placing the piers to the required depth, or shall cause the pier to drift from the required location. Dense natural rock or weathered rock layers shall not be deemed obstructions, and piers may be terminated short of design lengths on such materials. If the General Contractor cannot or does not remove such obstructions within one hour from the time Installer reports the obstruction to the General Contractor, the Installer may remove such obstructions with his own means. Should this occur, the Installer will be authorized to receive an extra to the contract to account for their additional expenses, including delay time involved to crew and equipment.

4.04 Bottom Stabilization Verification Test

After completion of the bottom pier bulb, or at anytime during the process of constructing the pier, the energy source may be turned off, and a bottom stabilization verification test may be performed. These tests shall be performed when a new soil formation is encountered, or at the beginning of a project to provide quantitative information on pier stabilization. A reference bar is placed over the cavity, and a mark is made on the tamper shaft that has been placed on top of the compacted aggregate. The energy to the tamper is restarted. If the measured vertical movement exceeds 150% of the value achieved during the load test, added energy is applied to redensify the bulb. The procedure for measuring is then repeated. If there is still movement greater than 150% of that achieved during the load test and greater than ½ inch, a lift of loose aggregate may be placed on top of the compacted aggregate, and the verification test may be performed on this next lift after it is densified. If there is excessive movement on this lift, another lift may be placed and tested. Movement must be limited to below 150% of the values achieved for the load test before completion of 2/3 of the pier depth unless unusually powerful modified hudraulic hammers are being used with tamper heads smaller than 26 inches in diameter..

4.05 Rejected Aggregate Piers

Aggregate piers improperly located or installed beyond the maximum allowable tolerances shall be abandoned and replaced with new piers, unless the Engineer approves other remedial measures. All material and labor required to replace rejected aggregate piers shall be provided at no additional cost to the Owner, unless the cause of rejection is due to an obstruction or mislocation.

4.06 Plan Location and Elevation of Aggregate Piers

The center of each pier shall be within six inches of the plan locations indicated. The final measurement for the top of aggregate piers shall be the lowest point on the aggregate in the last compacted lift. Piers installed outside of the above tolerances and deemed not acceptable shall be rebuilt at no additional expense to the Owner, unless mislocated by the General Contractor.

4.07 Footing Bottom

A. All excavations for footing bottoms supported by aggregate pier foundations shall be prepared in the following manner by the General Contractor: Over excavation below the bottom of footing elevation shall be limited to 3 inches. This includes limiting the teeth from excavators from over excavation beyond 3 inches below the footing elevation.

B. Compaction of surface soil and top of aggregate piers shall be prepared using a standard, hand-operated impact compactor ("Whacker Packer," "Jumping Jack," or equal). Compaction shall be performed over the entire footing bottom to compact any loose surface soil and loose surface pier aggregate.

C. Excavation and surface compaction of all footings shall be the responsibility of the General Contractor.

TABLE 4.2 - Preliminary Values for Geopier® Soil Reinforcement Design*

SPT = N Blows Per Foot All Soils	UCS, psf Fine-Grained Soils	Sands & Sandy Silts			Silts & Clays			Peat		
		Allowable Composite Footing Bearing Pressure, psf (q_{all})	Geopier® Element & Footing Segment Capacity, kips [1] (Q_{cell})	Geopier® Element Stiffness Modulus, pci [2] (k_g)	Allowable Composite Footing Bearing Pressure, psf (q_{all})	Geopier® Element & Footing Segment Capacity, kips [1] (Q_{cell})	Geopier® Element Stiffness Modulus, pci [2] (k_g)	Allowable Composite Footing Bearing Pressure, psf (q_{all})	Geopier® Element & Footing Segment Capacity, kips [1] (Q_{cell})	Geopier® Element Stiffness Modulus, pci [2] (k_g)
1-3	200-1000	5000	65	165	4500	50	125	3500	30	75
4-6	1001-2300	6000	90	225	5000	70	175	4000	45	110
7-9	2301-3500	7000	105	260	6000	85	210	5000	55	125
10-12	3501-4600	8000	115	285	7000	100	250	N/A	N/A	N/A
13-16	4601-6000	8500	125	310	7000	105	260	N/A	N/A	N/A
17-15	6001-8000	9000	130	325	7500	110	275	N/A	N/A	N/A
Over 25	Over 8000	10,000	145	360	8000	120	300	N/A	N/A	N/A

Notes:

1. For 18-inch Geopier® elements, multiply by 0.45
   For 24-inch Geopier® elements, multiply by 0.7
   For 36-inch Geopier® elements, multiply by 1.3
2. Geopier® element modulus to be confirmed by full-scale modulus test as determined by Geopier designer.

GEOPIER
FOUNDATION COMPANY