R. C. KANNAN & ASSOCIATES, INC.
CIVIL, GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS
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July 5, 2006
Project No. 1908

Terra Excavating, Inc.
2812 Airport Road
Plant City, FL 33563

Mr. Nick Kotaiche/ Mr. George Farrell

Re: **Proposed Multi-family residential Development**
13400 Pine Street, Largo, FL

SUBSURFACE EXPLORATION AND FOUNDATION DESIGN

Dear Mr. Katcha/Mr. Farrell:

We had completed the soil borings at the above property for the proposed development of the parcel in October 2004 and submitted a preliminary report. Based on further discussions and meetings with foundation specialty contractors and Palm Harbor homes we have evaluated the foundation design needs and provide this report for site preparation and foundation design.

This report presents a proposal for site preparation using a technique called surcharging or pre-loading. This requires applying the building load over a period of time to induce settlements that would have occurred under the building loads.

The 18-acre site has been extensively excavated to obtain fill material and has been backfilled with construction debris for at least three decades. Substantial amount of such debris was encountered in most of the borings and would present a challenging situation for foundation design. As we pointed out earlier, it is often necessary to "design as you go" in view of the uncertainties in the subsurface conditions. We would continue to work with you at every stage from concept to completion.

We appreciate this opportunity of providing this service to you. If you have any questions or when we may be of further assistance, please call us.

Sincerely,

Ramanuja C. Kannan, P.E.
Florida Registration No. 38688

7421 – 114th Avenue North, Suite 203, Largo, FL 33773-5199
PHONE: (727) 548 8080 FAX (727) 548 1978

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1.0 SCOPE OF WORK

Fieldwork was completed between October 25 and 28, 2004. Twenty (20) Standard Penetration test borings were conducted at the approximate locations of the proposed townhome, and a proposed retention pond for this tract. Data from the field exploratory program was used in the preliminary site planning, sizing and design of the proposed retention area and buildings.

This report provides a method of foundation soil stabilization and recommendations for the design of foundations.

2.0 FIELD WORK

The fieldwork consisted of performing twenty (20) SPT borings at the approximate locations shown in the attached sketch. These were located in the field by our field crew, by referencing to property boundaries and other prominent landmarks shown on an aerial photograph using a tape measure. The locations were approximate at the time of the fieldwork, but the borings were located within the footprint of the proposed buildings. The borings were terminated at an approximate depth of 30 feet below the ground surface, though some had to be taken to a depth of about 50 feet to verify the extent of the debris.

3.0 LABORATORY TESTS

The samples obtained from the field exploration were examined by a geotechnical engineer in our laboratory for further identification and to determine if additional tests are necessary. The soils were classified according to the Unified Soil Classification System (ASTM D-2487-83). At this time no additional tests were considered necessary.

4.0 SITE SOIL CONDITIONS

The subsurface soil conditions vary depending upon the depth of the sand excavated. However, a generalized soil profile is developed here for the purpose of considering the probable design constraints for various engineering applications. Boring logs included in the preliminary report of October 2004 show the variation between the borings. The soil stratigraphy may be described as follows for the purposes of foundation design:

From	To	Soil description
Surface	10 feet	Poorly graded sands (SP), loose to medium dense, Soil cover mixed with debris such as clay, concrete, limerock and brick fragments.
10 feet	30 feet	Sand with silt or clay (SP-SM, SP-SC) to Silty clayey sand (SM/SC), mixed with wide variety of debris ranging from wood, stone, concrete, brick, plastic, steel cables and rebars, shell, asphalt, pieces of construction equipment etc. Occasional cemented sand and sandy clay lenses in very hard to very dense state (refusal). Lost circulation in debris at most locations. Occasionally sampler advances by the weight of rods or the weight of hammer.
30 feet	55 feet	Stratum consisting of mixtures of Silty sand, silty clay, sandy clay and clayey sand, considerably lesser debris. Stratum in very variable condition ranging from extremely loose to very dense or hard state. Lost circulation of drilling fluid and encountered sampler advance by weight of hammer or weight of rods at some locations between 30 and 45 feet depth.

The ground water table at some locations was encountered at an approximate depth of about 6 feet. When the debris is encountered and loss of circulation occurs, ground water table is difficult to record.

6.0 DESIGN DATA FOR BUILDINGS

Based on the fieldwork completed to date, the following design parameters are recommended for the various aspects of design.

6.1 Project Description

The project consists of 207 dwelling units in 35 buildings. The residential units are proposed to be pre-fabricated homes. At his time the homes supplied by Palm Harbor Homes are being contemplated and the foundation analysis is based on the loads provided by the manufacturer. The individual modular units vary in size from 1,280 square feet to 1,500 square feet and would be mixed in groups of 5 to 6 units per building. The units are approximately 30 feet wide and 48 feet deep. Some of the end units might have an attached garage, while the other (interior) units would have open parking. The individual units are two-storied structures. The number of units and buildings are summarized in the table below:

Building Type	Number of units	Overall dimensions of the building	Number of buildings	Total units
Type I	2 units 1,280 sq. ft. each 2 units 1,400 sq. ft. each 2 units 1,500 sq. ft. each	139' 4" by 30'	21	126
Type 2	4 units 1,400 sq. ft. each 2 units 1,500 sq. ft. each	143' 4" by 30'	11	66
Type 3	3 units 1,400 sq. ft. each 2 units 1,500 sq. ft. each	120' 0" by 30'	3	15
	Total units			207

The individual unit sizes are as follows:

Floor area	Manufactured size - overall	Number of units
1,280 sq. ft.	21' 4" by 30' 0"	42
1,400 sq. ft.	23' 4" by 30' 0"	95
1,500 sq. ft.	25' 0" by 30' 0"	70
	Total units	207

According to the manufacturer and the current requirements of Florida Department of community Affairs, these structures require a continuous stem wall foundation or may be "stilt set" if designed by an Registered Professional Engineer. Depending upon soil conditions, the units might be placed on spread footings designed to an allowable bearing capacity of 2,000 pounds per square foot.

6.2 *Foundation Design*

The information on soil engineering properties from the borings conducted could be used for the preliminary design of foundations for the proposed buildings at site. Because of the existing ground elevations, most of the buildings would be built on an engineered fill, estimated to be about 1 to 2 feet above the existing ground surface elevations. Some of the fill required might be obtained in-situ by site grading and the soil excavated from the various ponds. Sandy soils appear to be usable as fill, though the compaction characteristics will differ, depending upon the fines content. Additional soil borings at the proposed building locations and additional laboratory tests are necessary for an engineering evaluation for foundation design.

As most of the buildings will be placed on fill, the foundation design parameters would depend on achieving adequate compaction, as specified in a following section.

6.3 *Soil Improvement by surcharging (pre-loading)*

A number of different soil improvement techniques were evaluated as the subsurface soil conditions are highly irregular and extend as far as 50 feet below the ground surface. For lightly loaded structures such as 2 to 3 story buildings, it appears that stem walls could be, provided that the subsurface soils are compacted and/or otherwise improved to provide a uniform subgrade. Soil by improvement by CSV stabilization and the use of ductile iron pipe as an alterative foundation were considered. Both systems were determined to be uneconomical.

We therefore recommend the use of surcharging or pre-loading the building areas. In this method, the anticipated settlements that the buildings might undergo are artificially induced by loading the subsurface soils. As the subsurface soils are mostly granular, settlements could be induced by superimposing the weight of the completed building over the entire footprint of the building. Estimated settlement of about 1 inch could be induced by a fill height of about 4 feet to simulate the weight of the building.

In this option the entire building area would be surcharged (or pre-loaded) with soil or aggregate to simulate the effect of applying the building load. The surcharge load would induce settlements in the subsurface soils, which would then be removed and replaced with the building. As it would not be practical to surcharge all the buildings over the entire site simultaneously, this operation could be done in a rolling pattern, surcharging four or six buildings at a time. The surcharge load would be moved progressively from one group of building pads to another.

The constraint in using surcharge is the time it would take to induce settlements. If the load of the building is applied as surcharge, settlements should be monitored at monthly intervals. Pre-loading operation could be suspended if the incremental settlement between two consecutive readings is less than 10%. If the surcharge height is increased, the time required to induce the necessary settlements could be reduced and the building construction could commence at a shorter time. This requires further analysis and we would also conduct settlement studies on a test section by placing the surcharge load.

The following procedure is recommended:

- Clear the building area plus a margin of 15 feet to be surcharged.
- Level the exposed surface to cover the building area plus the margin cleared.
- Install at least 5 settlement plates (or electronic settlement gages) on each building pad, within the footprint of the building at various depths ranging from 2 feet below the estimated foundation elevation to 20 feet below the foundation elevation. Settlements at the center of the fill would be the highest and are the settlements at the corners would be

the least.

- Backfill the excavated area with suitable structural fill placed in one-foot lifts for a total height of 5 feet above the finished grade, sloping the fill to maximum of 1:3 slopes.
- Monitor the settlement over a period of 4 to 6 months and observe the increment over each previous reading. If the settlement increment between any two consecutive readings is less than 10%, the loading operation might be stopped.
- Once it is determined that adequate settlements have been induced, the fill might be removed and moved to another building footprint.
- After the fill is removed, the building footprint should be excavated to the bottom of the footing elevation and compacted to achieve a minimum of 95% of the Modified Proctor Maximum density (AASHTO T-180 method). Any unsuitable material encountered during this operation must be over-excavated and backfilled with clean structural fill.
- A layer of geotechnical fabric should be provided. Tensar Geogrid or similar material may be used.
- The rest of the building pad can then be built up in one-foot compacted layers using clean structural fill to a minimum of 95% Modified Proctor maximum density.
- The foregoing operations must be completed under the supervision of a Florida registered Professional Engineer with knowledge and experience in surcharging, settlement monitoring and use of geotechnical fabrics.

After these operations are completed the footings as proposed may be designed for a net loading intensity not exceeding 2000 pounds per square foot (100 kN/m^2). The soil conditions at the footing elevation might not be the same for all buildings, or even within the footprint of the same building. If highly plastic clays or sandy clays are encountered at the footing elevation, these must be over-excavated and backfilled with clean granular fill to a minimum depth of 2 feet below the foundation elevation.

6.4 Foundation Design

The finished floor elevation must be set at an elevation of at least 16 inches (400 mm) from the bottom of the footing. The footing width must be a minimum of 20 inches (510 mm).

7.0 CONCLUDING REMARKS AND RECOMMENDATIONS

The recommendations of this report are based on field tests conducted in October 2004. The data obtained from the fieldwork and data provided by Palm Harbor Homes was used in the analysis. Recommendations for various elements of the project are based on assumptions stated in the various sections of the report. If these are different or changed at a later date, a review of our recommendations and revisions may be necessary.

Technical specifications for construction and detailed drawings could be provided during the construction stage if you require, by our office.

Please contact our office if the assumptions are not valid. In addition, during construction, if conditions other than those stated here are observed, these must be brought to our attention to verify that our recommendations are still valid. If necessary, based on a review of the new information, the recommendations of this report may be modified or additional controls specified.

We would recommend that additional soil borings be conducted at the locations of the proposed buildings, ponds, road and other improvements. In addition, shallow borings to a minimum depth of six feet should be conducted for all minor roadways. During construction, muck probing would be required, if any muck is encountered.

To determine the engineering properties of fill materials, laboratory tests on soil samples will be required. Laboratory tests for consolidation characteristics may also be required for medium to hi-rise buildings.

PROPOSED MULTI-FAMILY RESIDENTIAL DEVELOPMENT TERRA EXCAVTING PROPERTY PINE STREET AT 134th AVENUE NORTH LARGO, FL

Report Prepared for

Terra Excavating, Inc.
2812 Airport Road
Plant City, FL 33563

Mr. Nick Katcha/ Mr. George Farrell

Prepared by

R. C. Kannan & Associates, Inc.
P. O. Box 7525
Seminole, FL 33775

10/29/04

Ramanuja Chari Kannan, P. E.
Florida Registration No. 38688

Project No. 1908
October 29, 2004

R. C. KANNAN & ASSOCIATES, INC.
CIVIL GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS
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October 29, 2004
Project No. 1908

Terra Excavating, Inc.
2812 Airport Road
Plant City, FL 33563

Mr. Nick Katcha/ Mr. George Farrell

Re: **Proposed Multi-family residential Development**
13400 Pine Street, Largo, FL

SUBSURFACE EXPLORATION AND FOUNDATION DESIGN
PRELIMINARY FINDINGS

Dear Mr. Katcha/Mr. Farrell:

We have completed the soil borings at the above property for the proposed development of the parcel. The fieldwork was completed in line with our proposal dated October 20, 2004, and as authorized by you. As the final location of the structures and other improvements may vary slightly from the initial concept, the fieldwork covered in this scope of work is limited to the current conceptual plans. A more detailed investigation would be necessary at the location of the proposed building structures and other engineered facilities at a later date. In the mean time, we are forwarding this preliminary report to provide you with pertinent data to initiate your planning and design process.

This report presents the results of field exploratory borings. Our report presents a preliminary assessment for design purposes and is completed in line with generally accepted geotechnical engineering practice. No other warranty is implied or made.

The 18-acre site has been extensively excavated to obtain fill material and has been backfilled with construction debris for at least three decades. Substantial amount of such debris was encountered in most of the borings and would present a challenging situation for foundation design. We have proposed some foundation alternatives, which would be subject to further review and revision after the loading conditions are available. In situations like this, it is often necessary to "design as you go" in view of the uncertainties in the subsurface conditions. We would be pleased to work with you at every stage from concept to completion.

P. O. Box 7525 Seminole, FL 33775-7525 PHONE: (727) 548 8080 FAX (727) 548 1978

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1.0 SCOPE OF WORK

Fieldwork was completed between October 25 and 28, 2004. Twenty (20) Standard Penetration test borings were conducted at the approximate locations of the proposed townhome, and a proposed retention pond for this tract. Data from the field exploratory program could be used in the preliminary site planning, sizing and design of the proposed retention area and buildings. The following scope of work was included in this stage of fieldwork:

- Conducting Standard Penetration test (SPT) borings were completed at twenty (20) locations, as shown in the location sketch in the Appendix. All the borings were located within the probable pond and building areas based on a conceptual plan.
- Classification of the soils by a geotechnical engineer in the laboratory to estimate the engineering properties and determine the need for further laboratory tests.
- Prepare a preliminary engineering evaluation report giving recommendations for the design and for future detailed investigations.

2.0 FIELD WORK

The fieldwork consisted of performing twenty (20) SPT borings at the approximate locations shown in the attached sketch. These were located in the field by our field crew, by referencing to property boundaries and other prominent landmarks shown on an aerial photograph using a tape measure. The locations shown are hence accurate only to the extent possible by the method used. The borings were terminated at an approximate depth of 30 feet below the ground surface, though some had to be taken to a depth of about 50 feet to verify the extent of the debris. The equipment and procedures used in the SPT borings is described in the Appendix. Samples were packed and transported to the laboratory for an engineer's review and soil classification. Samples are normally retained for 30 days and discarded thereafter unless advised otherwise.

3.0 LABORATORY TESTS

The samples obtained from the field exploration were examined by a geotechnical engineer in our laboratory for further identification and to determine if additional tests are necessary. The soils were classified according to the Unified Soil Classification System (ASTM D-2487-83). At this time no additional tests were considered necessary.

4.0 GEOLOGY OF THE AREA

4.1 Regional Geology

The site covered under this study is shown in the Clearwater Quadrangle mapped by the United States Geological Survey. This map shows the site as a "sand pit." A number of U. S. Geological Survey and Florida Geological Survey publications are available, which discuss the various aspects of geology and the ground water sources in Pinellas County.

The surficial deposits of this part of Pinellas County consist mainly of a thin sand cover (varies from 10 to 40 feet in thickness) underlain by a stiff clay stratum, over late Pleistocene formations and recent deposits of Hawthorne Formation and Tampa[1] Limestones. Sea level changes in the Pleistocene Epoch have resulted in a number of solution cavities in the limestone formations. During the low sea level periods, the surface drainage had formed solution cavities in the limestones. During the periods of high sea levels, these cavities have been filled partially or fully with marine sediments, including a wide range of shells, silts, clay and sand. The upper reach of the Tampa Formation has thus become a porous limestone with numerous solution cavities. Any bedding planes or fractures that appear might have resulted primarily from the lithification of this limestone formation.

A cross-section of Pinellas County showing the most probable lithology at this site is described below. This cross-section is taken from a Bulletin [2] published by the Florida Department of Environmental Regulation.

The lithology may be described briefly as follows:

Dep
th Lithology

Surface	to	20/ 40 feet	Sand and Sand with silt or clay, recent deposits
20/40 feet	to	150 feet	Tampa and Suwannee Limestone Formations, Pleistocene to recent, with solution and erosion features.
150 feet	to	300 feet	Eocene formations of coquinoid limestone belonging to the Ocala Formation.
300 feet	to	1100+feet	Avon Formation of early Eocene Epoch, hard fossiliferous, Dolomitic Limestone

1 The limestones of Tampa and Suwannee limestones are not very easily distinguished in this area.

2 Edward A. Fernald and Donald J. Patton, Editors, Water Resources Atlas of Florida, Florida State University, 1984.

4.2 *Sinkhole Potential*

A number of paleosinks and relic sinkholes, which have, since been in-filled with debris can be seen as "lakes" in northern Pinellas County. The water elevation in these lakes also corresponds closely with the water elevation in the surface water (or ground water) aquifer. In this part of the Pinellas County, the frequency of sinkholes reported is low, though very few sinkhole-damaged homes have been reported within a five-mile radius of this area. A detailed geophysical investigation would be required to estimate the structural integrity of the limestone formation within the parcel. Published geological literature indicates that the separation between surface water aquifer and the Floridan Aquifer is not well defined. The site is located near a primary discharge area of Floridan Aquifer (towards the Gulf of Mexico), and there are a number of potential sources of potable water. This would imply that the surface water might migrate downward to the primary source of drinking water supplies for this area. When this migration occurs, there is also a potential for accelerating sinkhole formation. Hence the site should be considered to be in an area of active ground water migration. This will have to be considered in the design of the improvements, if it would alter the patterns of ground water flow.

4.3 *Hydrogeological Conditions*

The lithology described above is based on information on file with the U. S. Geological Survey in published bulletins. Of interest to this project are the surficial Sand (SP) and Sand with silt or clay (SP-SM or SP-SC) deposits near the surface and the upper reaches of the Hawthorne Formation. The surficial sand deposit is about 15 to 40 feet thick in this area. At this depth there is stiff clay to a depth of about 50 feet below the ground surface. This becomes a more gravelly clay stratum beyond this depth. This stratum represents the upper reaches of the Hawthorne formation and is relatively impervious as compared to the surface stratum. For all practical purposes, the upper 30 feet of soils may be taken as the surface (or water table) aquifer. However, as the site has been excavated and backfilled, the hydrogeological conditions could not be generalized. In general, construction debris tends to be highly porous because of the void spaces between the non-homogeneous components. The clayey soils of the Hawthorne Group isolate the solution cavity ridden limestones of the underlying Tampa Formation, a porous and highly permeable rock. In this part of Pinellas County, the clay stratum is sometimes discontinuous and the limestone formation is encountered between 50 to 120 feet below the ground surface at some locations. Therefore, there is a potential for interconnecting the surficial and the Floridan Aquifers. At this site, as the clay stratum varies in thickness. The site stratigraphy also suggests that the surface water and the Floridan aquifer may be undifferentiated at this site. The estimated elevation of the Upper Floridan Aquifer is approximately +20.00 NGVD in the vicinity of this site. This is very close to the ground water elevation of the surficial aquifer.

The surficial Sand (SP) and Sand with silt or clay (SP-SM or SP-SC) extends to an approximate depth of about 40 feet below the surface. This stratum has a very high permeability and our experience in this area shows that the permeability of this stratum is on the order of 10 to 30 feet per day, in its undisturbed state. The site of the proposed townhome complex appears to be classified as Myakka Fine Sand in the Soil Survey map. However, this sand has been excavated extensively.

Surface water drainage from the site is anticipated to follow the topography and would therefore be expected to have a slope from east to west. A northeast sector of the site might be subject to flooding, depending upon the elevations and the proximity of bodies of water. The ground water table is likely to fluctuate seasonally.

5.0 SITE SPECIFIC SOIL CONDITIONS

The subsurface soil conditions vary depending upon the depth of the sand excavated. However, a generalized soil profile is developed here for the purpose of considering the probable design constraints for various engineering applications. Boring logs included in the Appendix show the variation between the borings. The soil stratigraphy may be described as follows for the purposes of foundation design:

From	To	Soil description
Surface	10 feet	Poorly graded sands (SP), loose to medium dense, Soil cover mixed with debris such as clay, concrete, limerock and brick fragments.
10 feet	30 feet	Sand with silt or clay (SP-SM, SP-SC) to Silty clayey sand (SM/SC), mixed with wide variety of debris ranging from wood, stone, concrete, brick, plastic, steel cables and rebars, shell, asphalt, pieces of construction equipment etc. Occasional cemented sand and sandy clay lenses in very hard to very dense state (refusal). Lost circulation in debris at most locations. Occasionally sampler advances by the weight of rods or the weight of hammer.
30 feet	55 feet	Stratum consisting of mixtures of Silty sand, silty clay, sandy clay and clayey sand, considerably lesser debris. Stratum in very variable condition ranging from extremely loose to very dense or hard state. Lost circulation of drilling fluid and encountered sampler advance by weight of hammer or weight of rods at some locations between 30 and 45 feet depth.

The ground water table at some locations was encountered at an approximate depth of about 6 feet. When the debris is encountered and loss of circulation occurs, ground water table is difficult to record.

6.0 DESIGN DATA

Based on the fieldwork completed to date, the following design parameters are recommended for the various aspects of design.

6.1 Design of detention/retention ponds

The pond location for the proposed townhome complex is not finalized at this time. The most probable location for the pond is in the center of the property. It is anticipated to be surrounded by the proposed new buildings.

The ground water table is anticipated at an average depth of about 6 feet. The seasonal high water table for the design of a detention pond is difficult to determine, and therefore might be taken at 3 to 4 feet below grade at most locations. This is influenced primarily by the topography, existing drainage conditions and the subsurface soil conditions. The ground water table is normally encountered at about 1 to 3 feet lower than the seasonal high water elevation. The depth of the surficial aquifer for the purposes of design may be taken as 30 feet on the average.

6.2 Soil Permeability Data

The permeability of the surficial aquifer is estimated to be on the order of 5 to 20 feet per day (on the order of 1×10^{-3} cm/sec.). This is based on the soil classification and no field tests have been conducted as a part of this study. However, for aesthetic reasons, the storm water treatment ponds at this site might be designed as detention ponds.

6.3 Surface Drainage

Based on a review of the aerial photographs and US Geological Survey quadrangle map, the nearest drainage channel in this area is McKay Creek, located about 400 feet to the east of the site. There are roadside swale on the west and south boundary of the property currently channeling part of the surface water flow from the site and it appears to be hydraulically connected to McKay Creek. McKay Creek is part of the surface water control system that includes Taylor Lake, Walsingham Reservoir, Lake Seminole and Boca Ciega Bay. In the proposed development, some of the changed surface drainage conditions would have to be stored, diverted or discharged off-site.

6.4 *Pond Design Parameters*

The retention ponds may be designed with side slopes (ratio of horizontal to vertical) of approximately 3: 1 to 4: 1. As most of the side slopes for the proposed pond will be cut in sandy soils and debris, side slopes steeper than 30 degrees to the horizontal are not recommended. The type of soils encountered would limit the maximum depth of excavation. If necessary, a liner would be used to provide the required impervious slopes and bottom.

Depending upon the designed pond bottom elevations, dewatering should be anticipated. Most of the sandy material excavated from the ponds might be suitable for reuse as fill or landscape material, if the debris could be separated. Please contact our office for an evaluation of the excavated material for its potential use as fill. The engineering properties of the excavated material should be determined by additional laboratory tests.

7.0 FOUNDATION DESIGN FOR BUILDINGS

The information on soil engineering properties from the borings conducted could be used for the preliminary design of foundations for the proposed buildings at site. At this time no details of the proposed buildings are available. Because of the existing ground elevations, most of the buildings would be built on an engineered fill. Some of the fill required might be obtained in-situ by site grading and the soil excavated from the various ponds. Sandy soils appear to be usable as fill, though the compaction characteristics will differ, depending upon the fines content. Additional soil borings at the proposed building locations and additional laboratory tests are necessary for an engineering evaluation for foundation design.

As most of the buildings will be placed on fill, the foundation design parameters would depend on achieving adequate compaction, as specified in a following section.

7.1 Foundation Design: Two to Three Story, Low-rise Buildings

For lightly loaded structures such as 2 to 3 story buildings (buildings under four stories), it appears that post-tensioned mat foundations could be used, considering the fact that some structural fill will be required under these buildings. The subsurface soil conditions could be adequately prepared to support the masonry block load bearing walls, tilt-up construction or a concrete framed building on a spread footing system. Adequate site preparation is necessary and should be carried out to provide an engineered fill under the spread footings and the floor slab.

If spread footings are proposed, typically they may be designed for a net loading intensity not exceeding 2000 pounds per square foot (100 kN/m^2). However, in view of the variable soil conditions, substantial site preparation and subsurface soil stabilization would be required.

The soil conditions at the footing elevation might not be the same for all buildings, or even within the footprint of the same building. If highly plastic clays or sandy clays are encountered at the footing elevation, these must be over-excavated and backfilled with clean granular fill to a minimum depth of 2 feet below the foundation elevation.

7.2 Site Preparation

Building on filled sites is always a problem and most often such sites are developed as golf courses after placing adequate fill as cover. However, building on any soil condition is a matter of engineering and the only constraint is the value of the engineered product. In light of that philosophy we provide a few alternatives for stabilizing the subsurface soils for the proposed buildings.

7.21 *Excavate and Backfill*

Light residential, industrial and commercial buildings on mixed soil such as this site are often built under the erroneous assumption that the movement of construction equipment provides adequate compaction. The loading intensity of compaction equipment is very low and specific compaction equipment provide 2 to 3 times their compaction effort. Also, in sandy soils (SP, SP-SM, SP-SC type soils) vibratory rollers provide better compaction effort than smooth wheeled rollers. Thus adequate compaction by a heavy vibratory roller is essential to control settlements and to provide adequate bearing capacity.

Site preparation should be carried out and tested as specified, before construction of the structures begins. The specifications apply to the excavated ad backfilled soils, natural ground proof, and all fill placed on the site. The se specifications also apply to all cut surfaces, driveways and parking areas, where the fill height does not exceed 5 feet. Site preparation would include the following steps.

The unsuitable fill material must be excavated to a depth of at least 5 feet over all the building areas and backfilled with clean, granular fill, placed over two layers of geotechnical fabric. The procedure would approximately as follows:

- Excavate to a depth of 5 feet under the proposed building areas, plus a margin o 5 feet.
- Place a layer of geotechnical fabric such as Tensar or Miragrid.
- Backfill with a clean granular material such as crushed concrete product or limerock suitable for base course over the fabric for a depth of about 12 inches (300 mm). Compact this layer to a minimum of 95% of its Modified Proctor maximum density (AASHTO T-180). This material must meet the FDOT specifications for crushed aggregate (Section 204).
- Place a second layer of a woven geotechnical fabric over the compacted limerock (or crushed concrete).
- Backfill using a clean granular fill (such as SP. SP-SM or SP-SC or SW, SW-SM or SW-SC). The fines content (finer than 75 μm) should not exceed 10%. Compact backfill in one-foot layers. The compacted backfill should be carried to the finished floor elevation.
- The finished floor elevation must be set at an elevation of at least 16 inches (400 mm) from the bottom of the footing. The footing width must be a minimum of 20 inches (510 mm) and the bottom of the footing must be at least 60 inch (1.5 m) above the compacted limerock.
- The footings might be constructed as conventional footings using an allowable bearing capacity of 200 pounds per square foot (100 kN/m^2).
- Alternatively, post-tensioned mats could be used. If post-tensioned mats are used, the height of fill could be reduced by about a foot or more.

7.22 *Surcharging*

In this option the entire building area would be surcharged (or pre-loaded) with soil or aggregate to simulate the effect of applying the building load. The surcharge load would induce settlements in the subsurface soils, which would then be removed and replaced with the building. As it would not be practical to surcharge the entire site, this operation could be done in a rolling pattern, surcharging three or four buildings at a time. The surcharge load would be moved progressively from one group of building pads to another.

The constraint in using surcharge is the time it would take to induce settlements. If the load of the building is applied as surcharge, about 8 months to two years might be required to induce the necessary settlements. If the surcharge height is increased, this time could be reduced and the building construction could commence at a shorter time. This required further analysis and we would also conduct settlement studies on a test section by placing the surcharge load.

One of the ways of applying the surcharge loads in a rapid succession is to use dynamic compaction. In this method a heavy load, (about 5 to 30 tons) is dropped from a height of 10 to 30 feet over the area to be stabilized. The effect of dynamic compaction is to stabilize the subsurface soils to a depth greater than 10 to 30 feet depending upon the soil type. However, dynamic compaction would cause considerable nuisance and damage to the neighboring structures and preferably should not be used. Dynamic compaction is suitable for large isolated areas. After dynamic compaction is completed, the townhome buildings could be built on conventional spread footings. This is method of compaction is anticipated to be less expensive and far less time consuming than surcharging. A test section must be completed to verify that this method of compaction is a viable alternative.

7.23 *Deep foundations*

Deep foundations provide another option. A number of different types of piles are available, such as concrete, auger-cast and auger-displacement piles. Installation of all these pile types would require some pre-drilling. Pre-drilling would not only add to the cost of pile installation, but also would cause delays in view of the uncertain types of debris encountered. Some types of debris such as cables, rebars, etc. would make the installation of auger-cast piles difficult. Driven piles should preferably be designed to be larger (18-inch square) and for higher capacities (over 50 tons). The added cost of pile caps and grade beams should also be considered. If deep foundations were used, we would recommend that taller buildings (8 to 12 stories) be designed to take advantage of the pile foundations.

Small-diameter and lighter capacity piles are unsuitable for the project as currently envisaged, with lighter two to three story structures.

8.0 PAVEMENT DESIGN

8.1 Pavement Section

The major paved and parking areas proposed on this project consist of internal driveways subject to light loading. Heavier loading is anticipated in areas for truck-access, driveways and acceleration/deceleration lanes meeting Florida DOT[3] requirements, internal access roads for delivery, shipment and waste collection. Different pavement sections as recommended below may be used. For the paved areas, an asphalt pavement or a concrete pavement may be used. A recommended pavement section may be as follows, which is taken from our work on similar projects:

Subgrade	6" (150 mm)	Stabilized natural ground, compacted to a minimum of 98% of the Modified Proctor maximum density (ASTM D-1557). or Minimum LBR = 40 or FBV =75 psi.[4]
Base	6" (150 mm)	Soil cement base,[5] or equivalent pre-mixed soil cement base, with minimum 7 - day strength of 300 psi. Soil cement strength over 400 psi is not desired and should be avoided.[6]
Asphalt	1.75" (45 mm)	Asphalt pavement, Florida D. O. T. Type S-3.[7]

As an alternative to site-mixed soil cement base, pre-mixed soil-cement base delivered under trade names such as "Permabase", "Durarock" or equivalent may also be used. In any case, for soil-cement base minimum 7 days' strength of 300 psi. (2,000 kPa) and compacted to a minimum of 95 % of ASTM D-1557 maximum density may be used. The specifications should require that the strength of the soil cement base does not exceed 400 psi.

Placement of the base must be monitored by our representative during construction. When a pre-mixed soil cement base is used, our technician will sample the factory mix twice each day during placement. The supplier of Permabase should submit a mix design for our review and records. Density tests on subgrade and base should also be performed. Asphalt placement should be similarly tested to check compliance with Florida D. O. T. standards.

3 Department of Transportation

4 Florida Bearing Value. The LBR (Limerock Bearing Value) may be substituted for FBV at the discretion of the engineer, provided that the City/County/State requirements are met for roads and streets under their jurisdiction. Please see Florida D.O.T. Manual, Section 160 for details.

5 Please see Florida D.O.T. Manual, Section 270 for details.

6 Alternatively, In- situ "Limerock" base, compacted to a minimum of 98% of the Modified Proctor Maximum density (ASTM D-1557) may be used, where the base is at least 2 feet above the seasonal high water elevation at all times.

7 Florida D.O.T. Manual, Section 513. Equivalent County pavement specifications (if any) might also be used.

Alternatively, the following section may be used for internal driveways and parking areas:

Stabilization Fabric	Single layer	Mirafi Geolon HP 570, AMOCO 2044 Or any equivalent fabric meeting at a minimum, ASTM D 4595, ASTM D 4491, ASTM D 4751 and ASTM 4491 standards. The contractor must submit complete specifications of the substitute fabric for our approval. Fabric used must meet the.
Subgrade	18" (450 mm)	Stabilized natural ground or fill material, compacted to a minimum of 98% of the Modified Proctor maximum density (ASTM D-1557).
Wearing course	5" (125 mm)	Concrete pavement, designed with reinforcement and joints in accordance with PCI[8] pavement design standards,

For concrete curbs, the following stabilization is recommended:

Subgrade	12" (300 mm)	Stabilized natural ground or fill material, compacted to a minimum of 98% of the Modified Proctor maximum density (ASTM D-1557) or Florida D.O.T. Type B Stabilization, FBV = 75 psi.

For Quality Control, either the Project General Specifications or the Pinellas County testing frequency standards may be specified for type and frequency of tests required on pavements, utility trenches and backfill over utility cuts.

8.2 *Under drains*

As most of the pavement would be at elevations higher than the adjoining streets (Pine Street and 134th Avenue North), and surface drainage will be channeled through on-site retention areas, we do not anticipate the use of under drains for the pavements within this property. However, at this preliminary stage, we estimate that wherever the pavement section is at a finished elevation lower than the crown elevation of the surrounding roads, under drains would be required. In such cases, the under drains should be provided with a positive outfall. The under drain system may consist of pre-fabricated or built-in-situ type.

8 Portland Cement Institute. Equivalent AASHTO standards may also be used.

9.0 CONCLUDING REMARKS AND RECOMMENDATIONS

The recommendations of this report are based on field tests conducted in October 2004. The data obtained from the fieldwork was used for a preliminary analysis. Some information was obtained from work done by our firm earlier, in connection with the proposed development. Our report is preliminary in nature and was completed in general accordance with accepted geotechnical engineering practice and no other warranty is implied or made.

Recommendations for various elements of the project are based on assumptions stated in the various sections of the report. If these are different or changed at a later date, a review of our recommendations and revisions may be necessary. Please contact our office if the assumptions are not valid. In addition, during construction, if conditions other than those stated here are observed, these must be brought to our attention to verify that our recommendations are still valid. If necessary, based on a review of the new information, the recommendations of this report may be modified or additional controls specified.

We would recommend that additional soil borings be conducted at the locations of the proposed buildings, ponds, road and other improvements. In addition, shallow borings to a minimum depth of six feet should be conducted for all minor roadways. During construction, muck probing would be required, if any muck is encountered.

To determine the engineering properties of fill materials, laboratory tests on soil samples will be required. Laboratory tests for consolidation characteristics may also be required for medium to hi-rise buildings.

We appreciate this opportunity of providing this service to you. If you have any questions or when we may be of further assistance, please call us.

Sincerely,

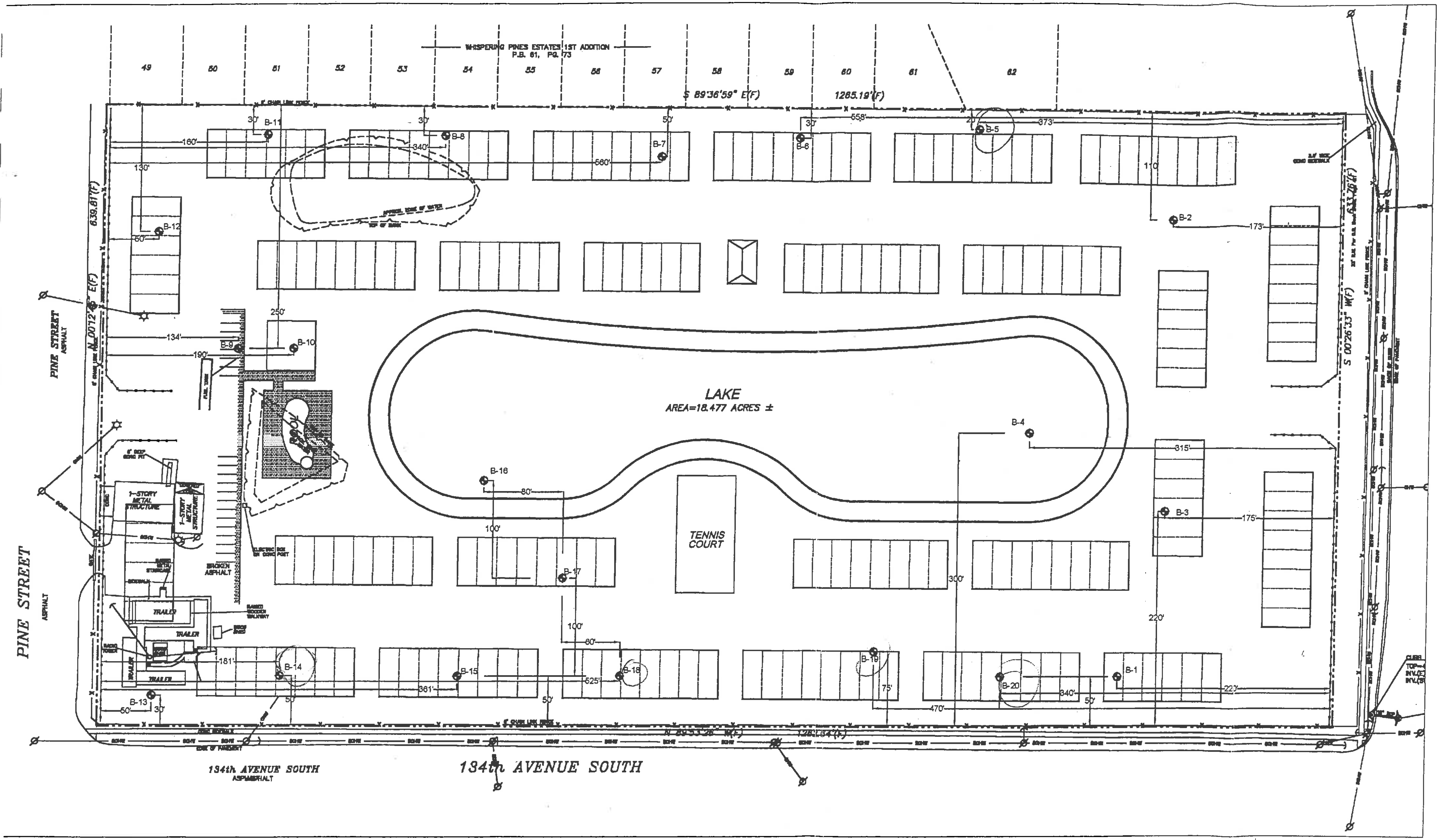
10/29/04

Ramanuja C. Kannan, P.E.
Florida Registration No. 38688

cc: Northside Engineering Services, Inc.: Mr. Houshang Ghoavee

APPENDIX

Boring Location Sketch



SOIL BORING LOCATION SKETCH

SCALE: 1" = 100'

LEGEND

B-1 APPROXIMATE LOCATION OF STANDARD PENETRATION TEST CONDUCTED BY R.C. KANNAN AND ASSOCIATES, INC. FROM 10/25-28/04.

2. SPTS WERE MARKED IN FIELD BY A REPRESENTATIVE OF R.C. KANNAN AND ASSOCIATES, INC. SITE PLAN VIEW WAS PROVIDED BY NORTHSIDE ENGINEERING.

3. THIS IS AN APPROXIMATE SKETCH. DO NOT SCALE.

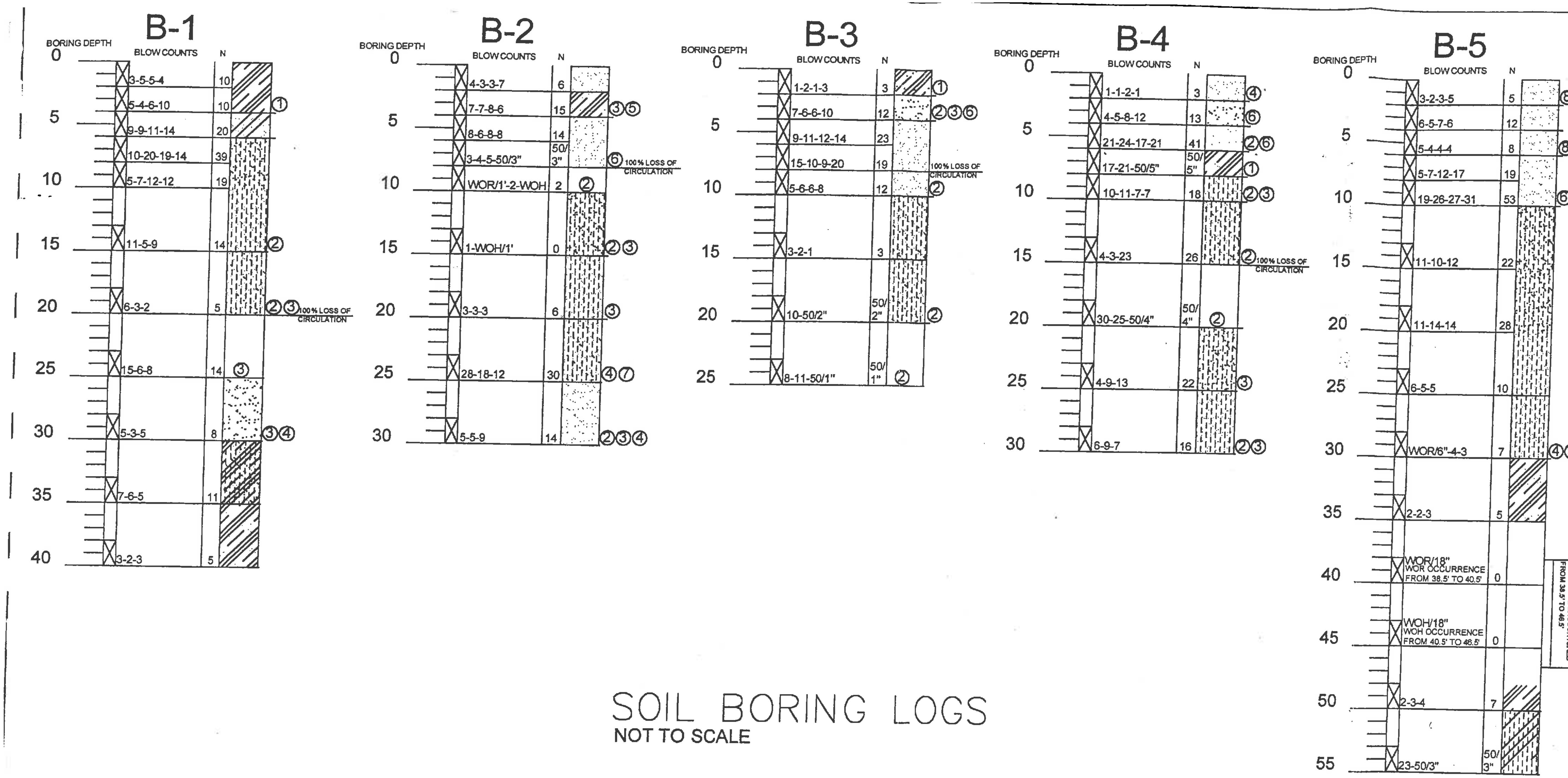
R.C. KANNAN and ASSOCIATES, INC.
7421 - 114TH AVE. N., SUITE 203
LARGO, FL 33773
PHONE: (727) 548-8080 FAX: (727) 548-1978
e-mail: rkannan@gate.net
web: www.gate.net/~rkannan

P-1

TERRA SITE TOWNHOMES
13400 PINE STREET
LARGO, FL 33774

SOIL BORING LOCATION SKETCH

JOB NO: 1908
dwg. DATE: 10/25-28/04
SCALE: AS SHOWN
by: FRS



SOIL BORING LOGS

NOT TO SCALE

NOTES

1. THIS IS A PRELIMINARY REPRESENTATION OF THE SUBSURFACE SOIL CONDITIONS BASED ON THE BORELOGS. THE STANDARD PENETRATION TESTS WERE CONDUCTED FROM 10/25-28/04. THE BORING LOGS ARE SUBJECT TO CHANGE.
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SAND (SP)
SAND w/ SILT (SP-SM)
SILTY SAND (SM)
SILT (ML)
CLAY (CL)
CLAYEY SAND (SC)
SANDY CLAY (CL)
SILTY CLAYEY SAND (SM-SC)
CLAYEY SILT (SC-SM)

1. W/ CLAYEY LIMESTONE
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4. W/ STONE FRAGMENTS
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7. W/ PLASTIC DEBRIS
8. W/ FIBROUS ROOTS
9. W/ SHELL FRAGMENTS
10. NO SAMPLE RECOVERED
11. W/ ASPHALT FRAGMENTS
12. W/ CEMENTED SAND FRAGMENTS
13. W/ CLAY NODULES

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P-2

TERRA SITE TOWNHOMES
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SOIL BORING LOGS

JOB NO: 1908
dwg. DATE: 10/25-28/04
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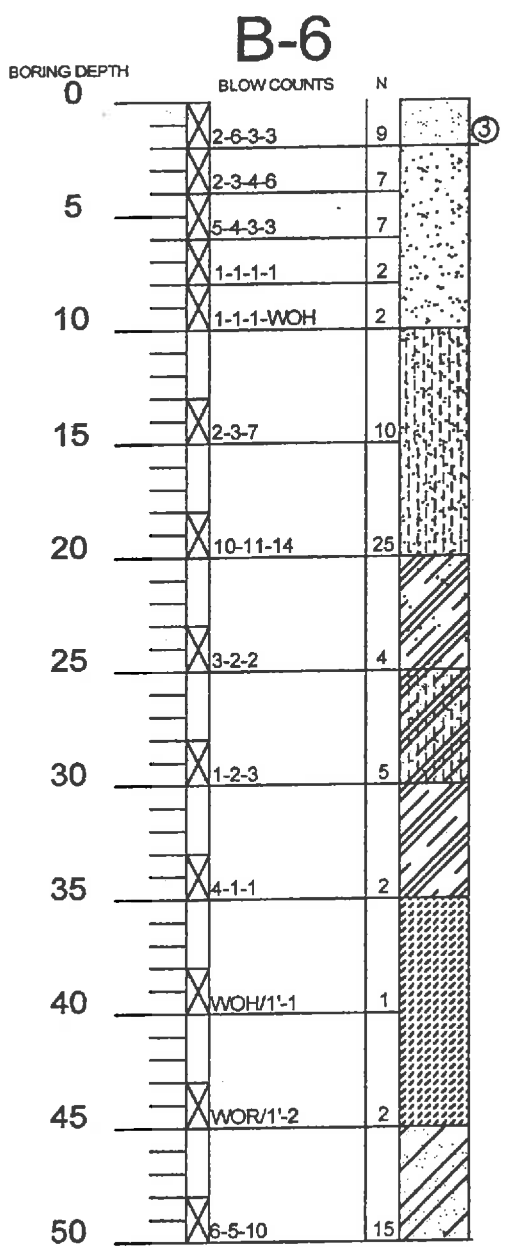

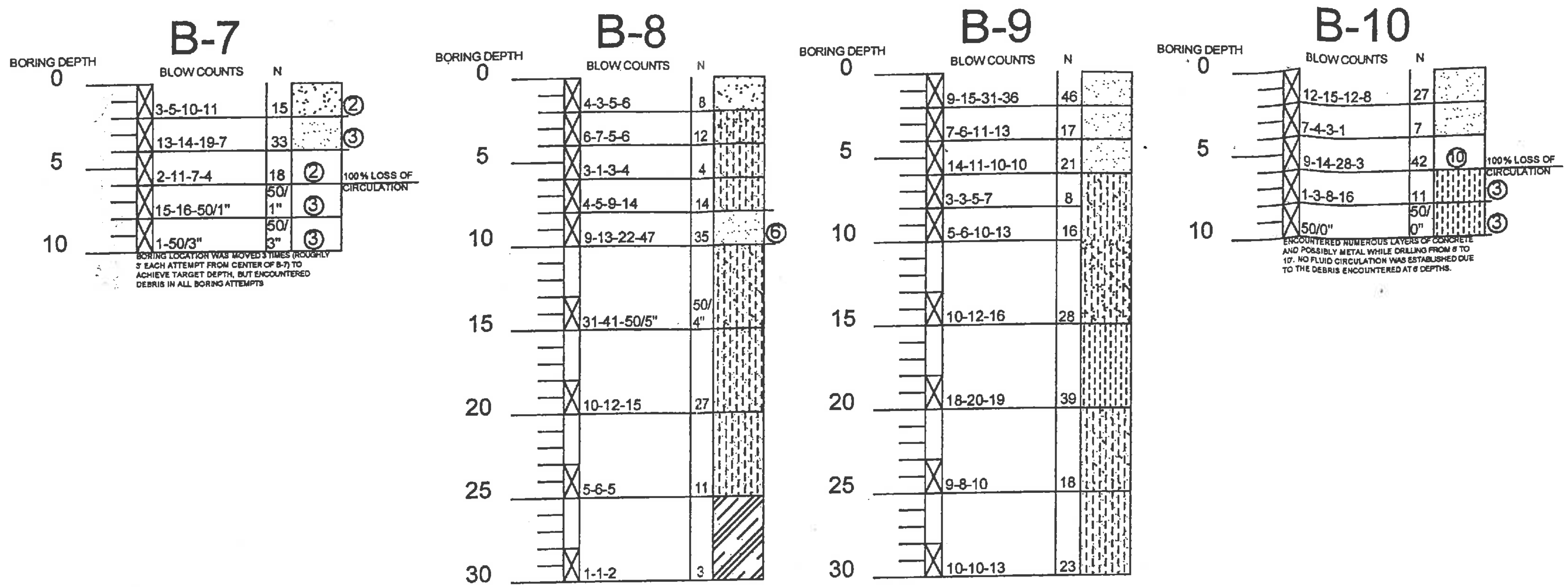

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9 W/ SHELL FRAGMENTS
10 NO SAMPLE RECOVERED
11 W/ ASPHALT FRAGMENTS
12 W/ CEMENTED SAND FRAGMENTS
13 W/ CLAY NODULES

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P-3

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SOIL BORING LOGS

JOB NO: 1908
dwg. DATE: 10/25-28/04
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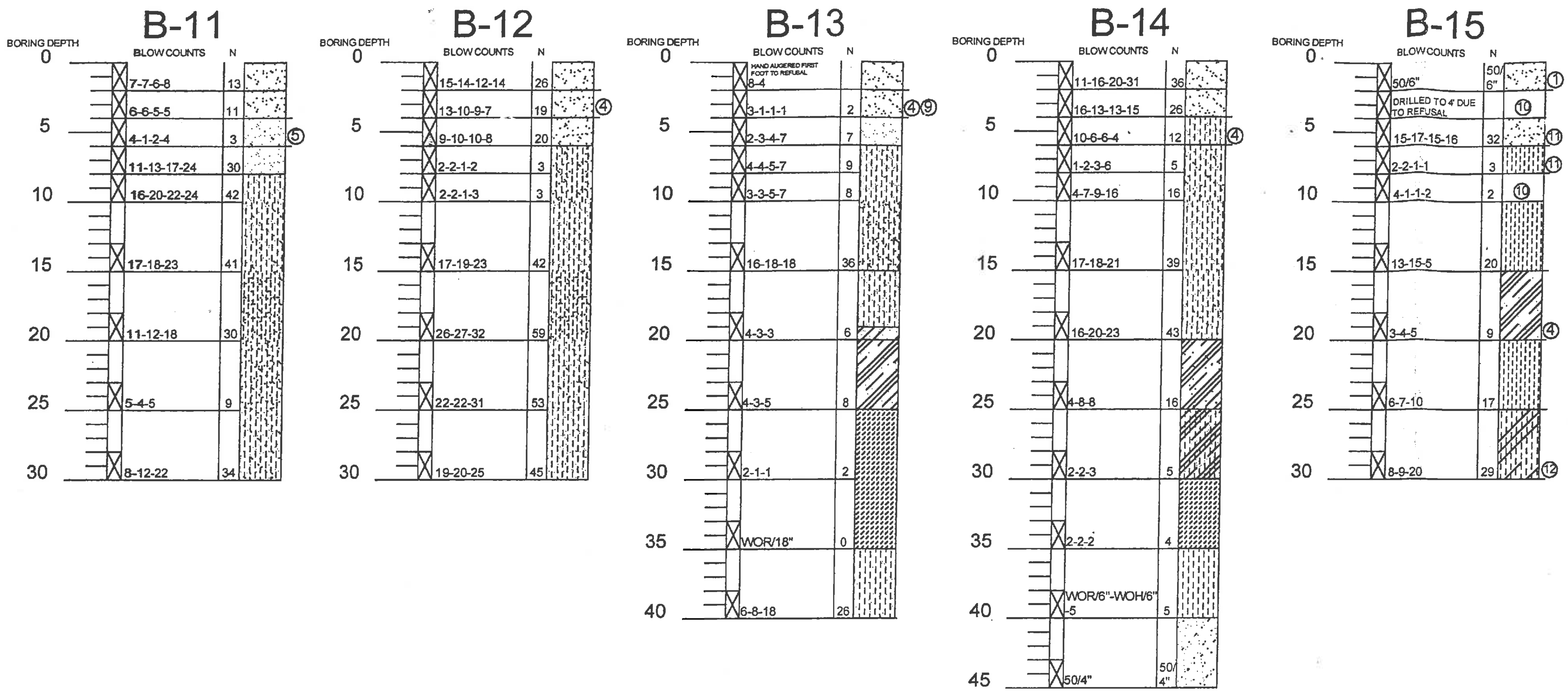

SOIL BORING LOGS

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P-4

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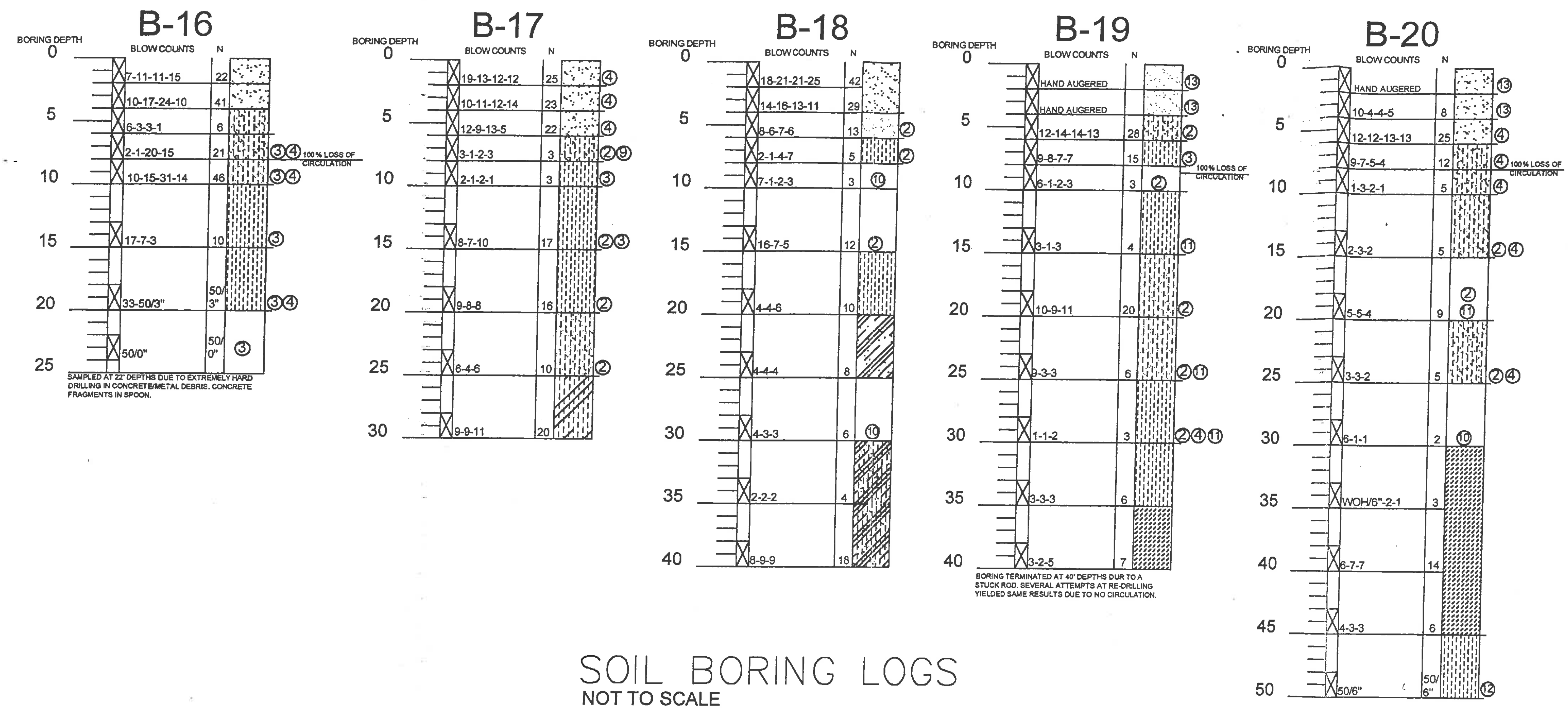

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P-5

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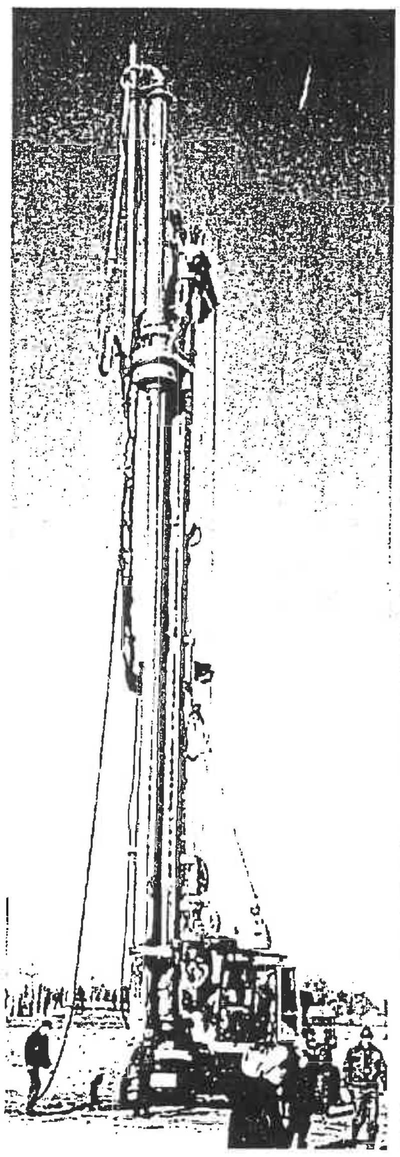
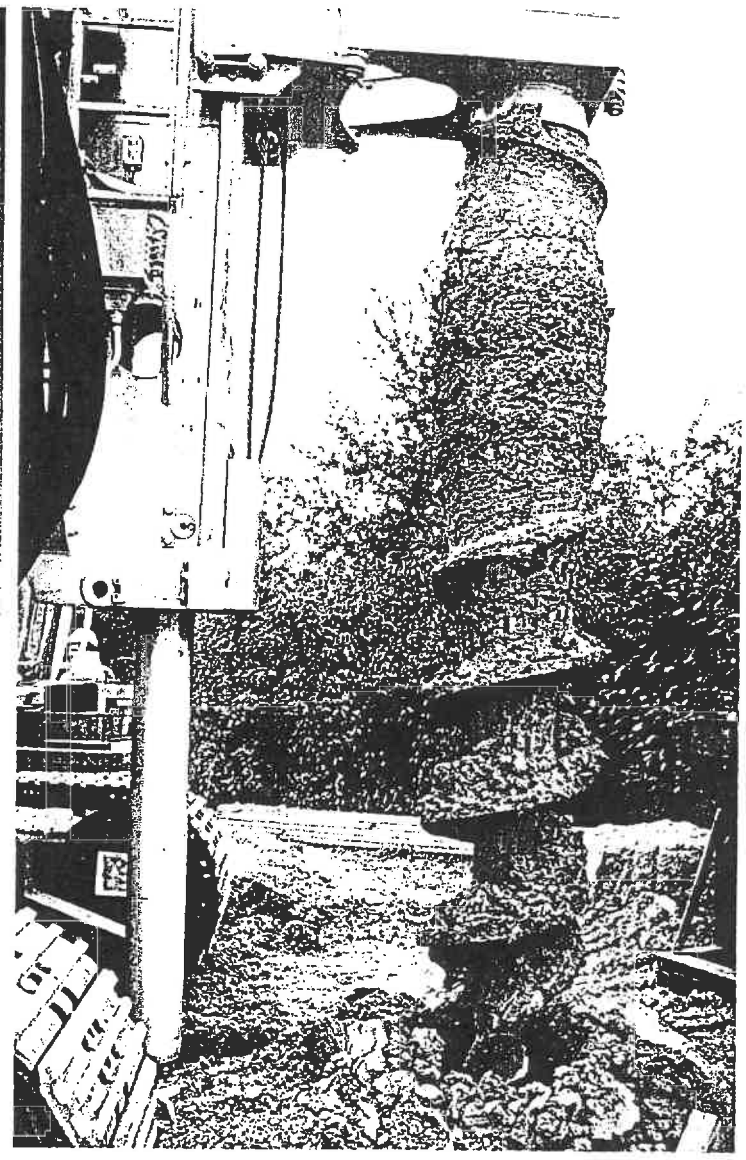
SOIL BORING LOGS

JOB NO: 1908
dwg. DATE: 10/25-28/04
SCALE: AS SHOWN
by: FRS

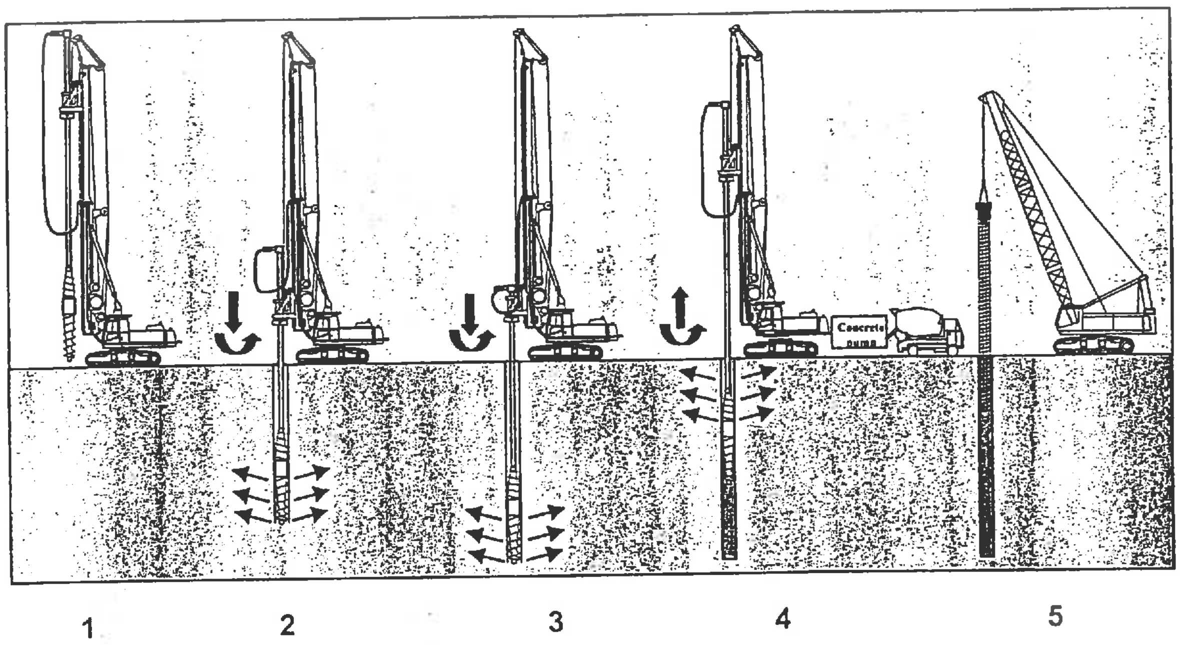
SDP
Soil Displacement Pile System
(BAUER BG-System)





BAUER

List of Contents

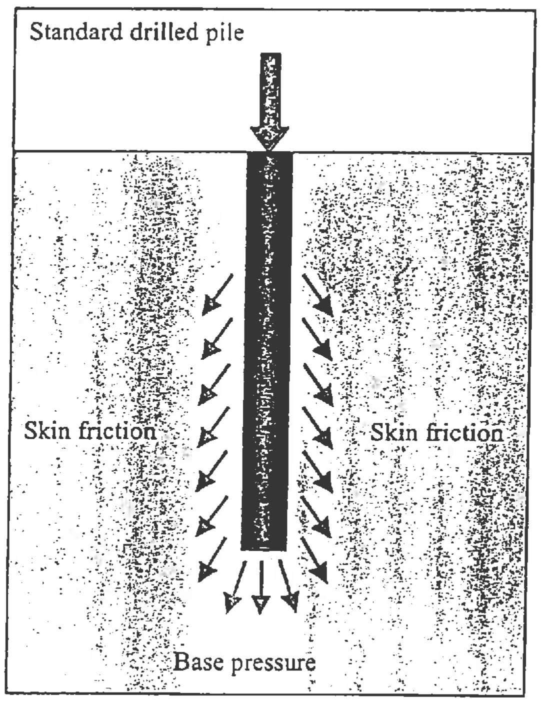


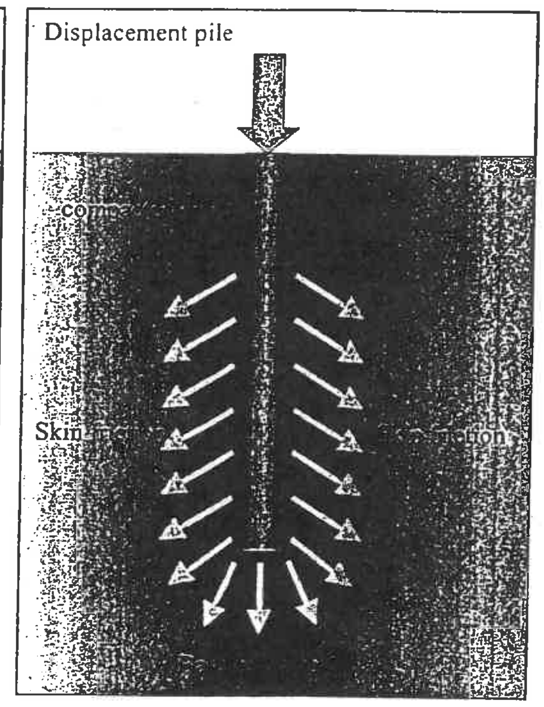
- **Step 1:** Positioning of rig
- **Step 2:** Lowering of the displacement tool into the ground by rotating and pushing of the tool. The soil is loosened by the auger starter and then pushed into the surrounding soil by the displacement body.
- **Step 3:** Installation depth can be extended up zo 10 m by using a kelly extension.
- **Step 4:** When reaching the final depth the tool is extracted and concrete is pumped through the hollow stem of the tool.
- **Step 5:** Installation of reinforcement cage (assisted by vibratory action) into the fresh concrete.

Advantages of the SDP System

High bearing capacity

- Displacement of soil volume into the surrounding soil creates a highly densified cylindrical soil volume.
- The load transfer area (densified soil volume) is increased by appr. 30 %.
- As a consequence thereof the skin friction is increased by appr. 30% and the base pressure is increased by appr. 50 – 70 % (in relation to the nominal diameter)





Vibration-free installation process

- Installation of the displacement tool with rotary drilling methods and applying of vertical crowd force .No vibrations or impacts on adjacent structures or buildings.

Avoiding of drilling spoil

- The soil is fully displaced into the surrounding soil mass during installation of the displacement tool.
- As there is no soil transported to the surface this piling system is ideally suitable for:

  - working in contaminated areas
  - excellent manoeuvrability on site
  - no soil loading and transport of excavated soil required

Advantages of the SDP System

High productivity

- The rate of productivity is dependent on

  -- Pile diameter
  – applied torque and crowd force
  – density (strength) of soil
  – displaceability of the soil
  – pumping capacity of concrete pump

Rates of productivity (estimated)			
Pile diameter	400 mm	500 mm	600 mm
torque (required)	150 kNm	220 kNm	250 kNm
crowd force (required)	20 to	28 to	35 to
non-cohesive soil, loose	6 m/min	6 m/min	6 m/min
non-cohesive soil, dense	3 m/min	2 m/min	1 m/min
cohesive soil, soft	6 m/min	6 m/min	6 m/min
cohesive soil, stiff	2 m/min	1 m/min	0,5 m/min
soft, fractured rock (socket)	0,1 m/min	0,1 m/min	0,1 m/min

Concreting speed (Pump capacity 30 m³/h)			
Pile diameter	400 mm	500 mm	600 mm
loose soil or soft consistency	3 m/min	2 m/min	1 m/min
dense soil, or stiff consistency	4 m/min	3 m/min	2 m/min

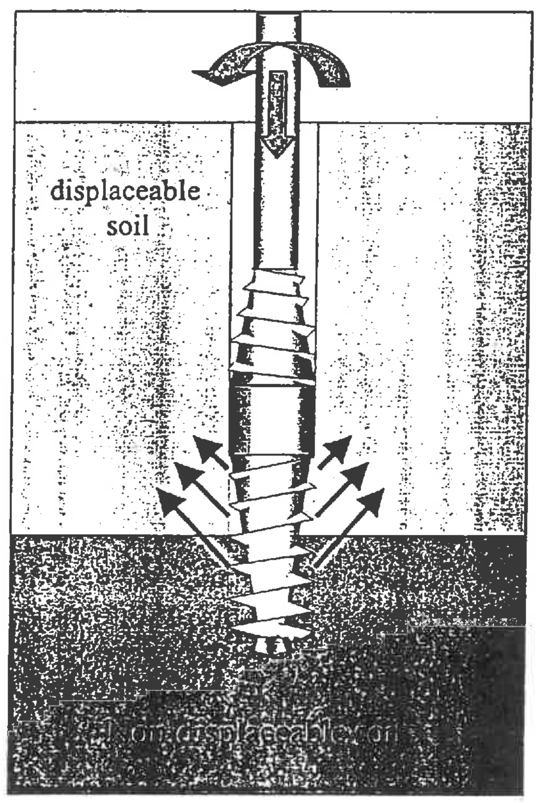
Minimizing of concrete consumption

- The overconsumption of concrete is considerably reduced as a result of the displacement procedure compared to other continuous flight auger techniques

Indicative values for excess concrete percentages			
Pile diameter	400 mm	500 mm	600 mm
soft, cohesive soil	1,25 *	1,22	1,20
loose, non-cohesive soil	1,15	1,15	1,15
dense, non-cohesive soil	1,10	1,09	1,08
stiff cohesive soil	1,0	1,0	1,0
* ratio installed volume / theor. concrete volume			

Reduced Production Cost

- The combination of high drilling speed,
  short concreting time (high pumping capacity)
  and short moving time (absence of excavated soil)
  results in daily performance rates which are superior to conventional drilling methods.
- Maximized performance and minimized equipment and staff input results in low opereational cost per linear meter of pile
- Low cost per tonne of applied load (due to increased bearing capacity as a result of the displacement effect)



SDP-system in "non-dispaceable" soil

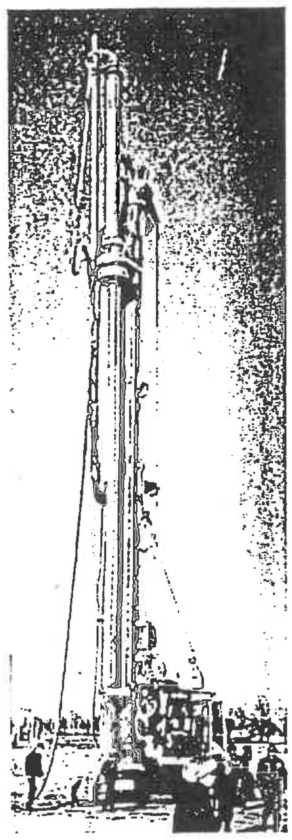
Non-displaceable soil is defined as:

-- dense non-cohesive soil (sand, gravel)
– hard cohesive soil (clay, silty clay)
– weathered, fractured rock

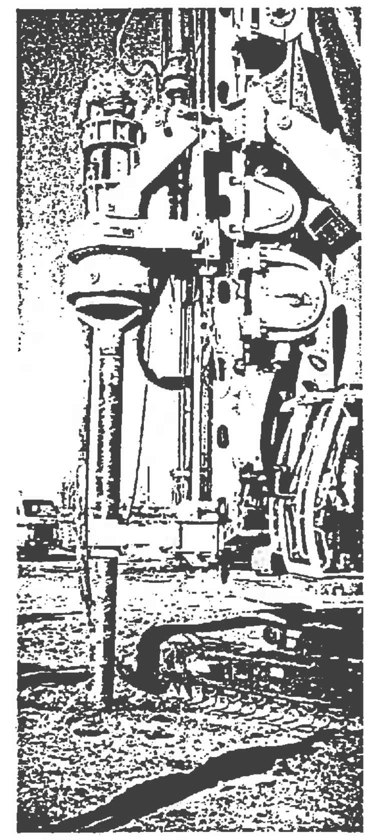
even in non displaceable soils the SDP system can be used under the following conditions:

- limited thickness of the non displaceable soil layer
- the strata above this layer have to be displaceable. When drilling into the non displaceable layer the excavated material is transported by the shape of the tool into the upper layer

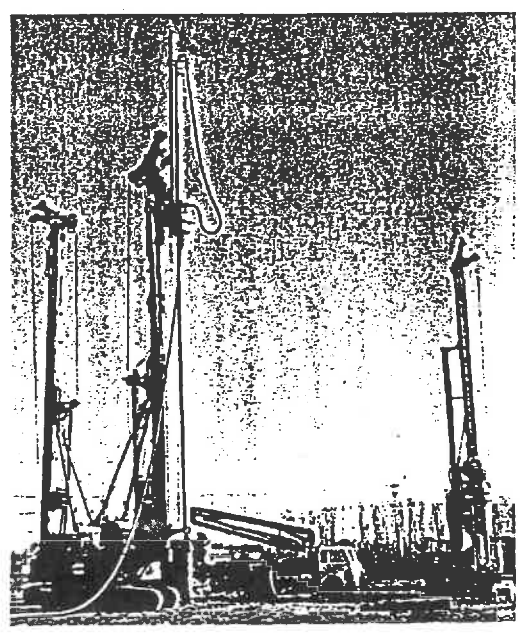
BG 18 and BG 22



BG 22
(with kelly extension)

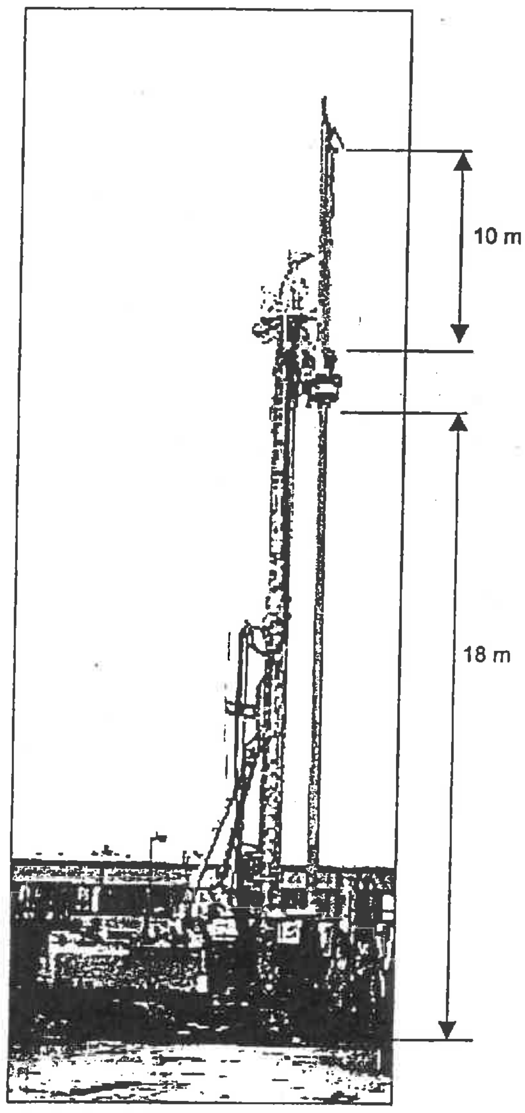


BG 18H
(without kelly extension)

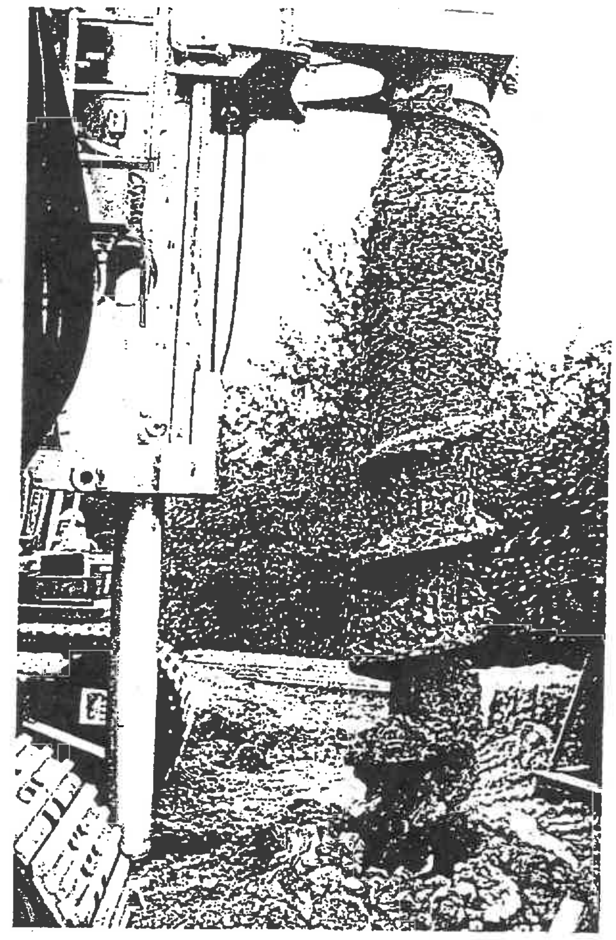


BG 18H and BG 22 (Fa. Jägerbau, Hungary)

BG 25

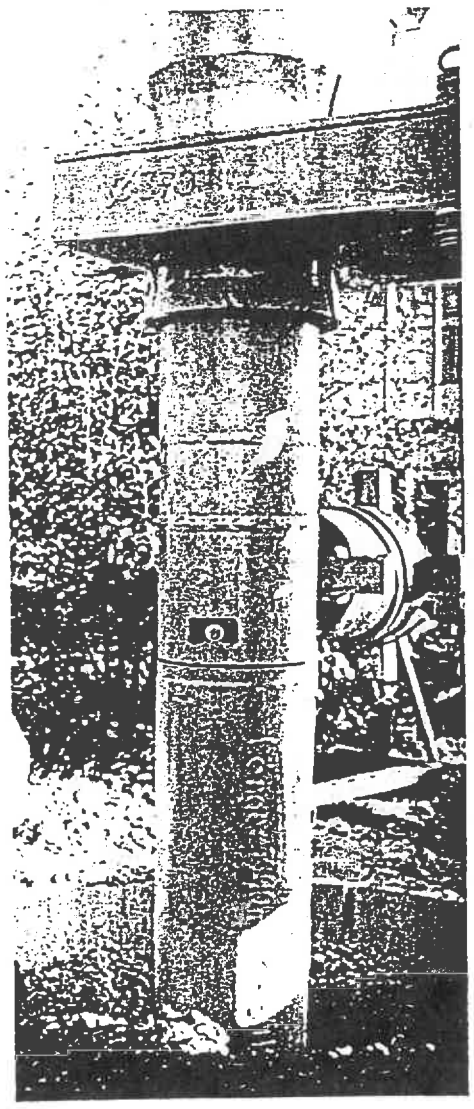


BG 25 for depth 28 m
(10 m extension)

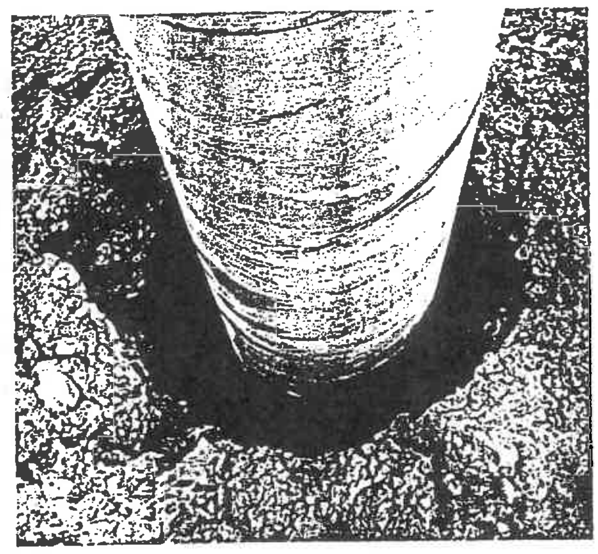


Displacement tool dia. 600 mm

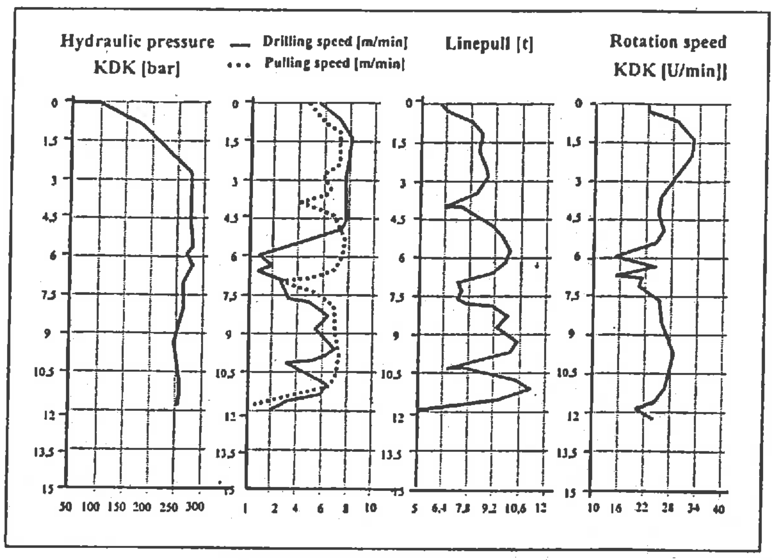
Drill Rods



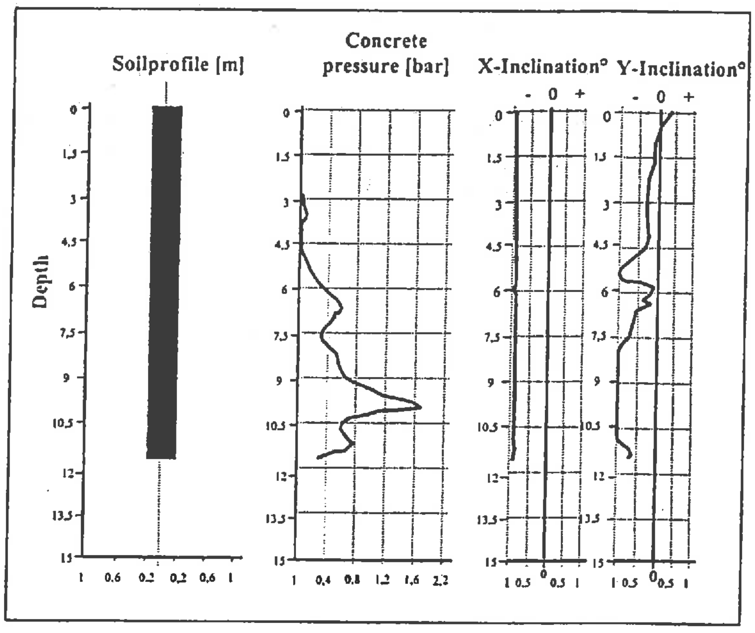
Drill rod dia. 350 mm
(with rod guide)



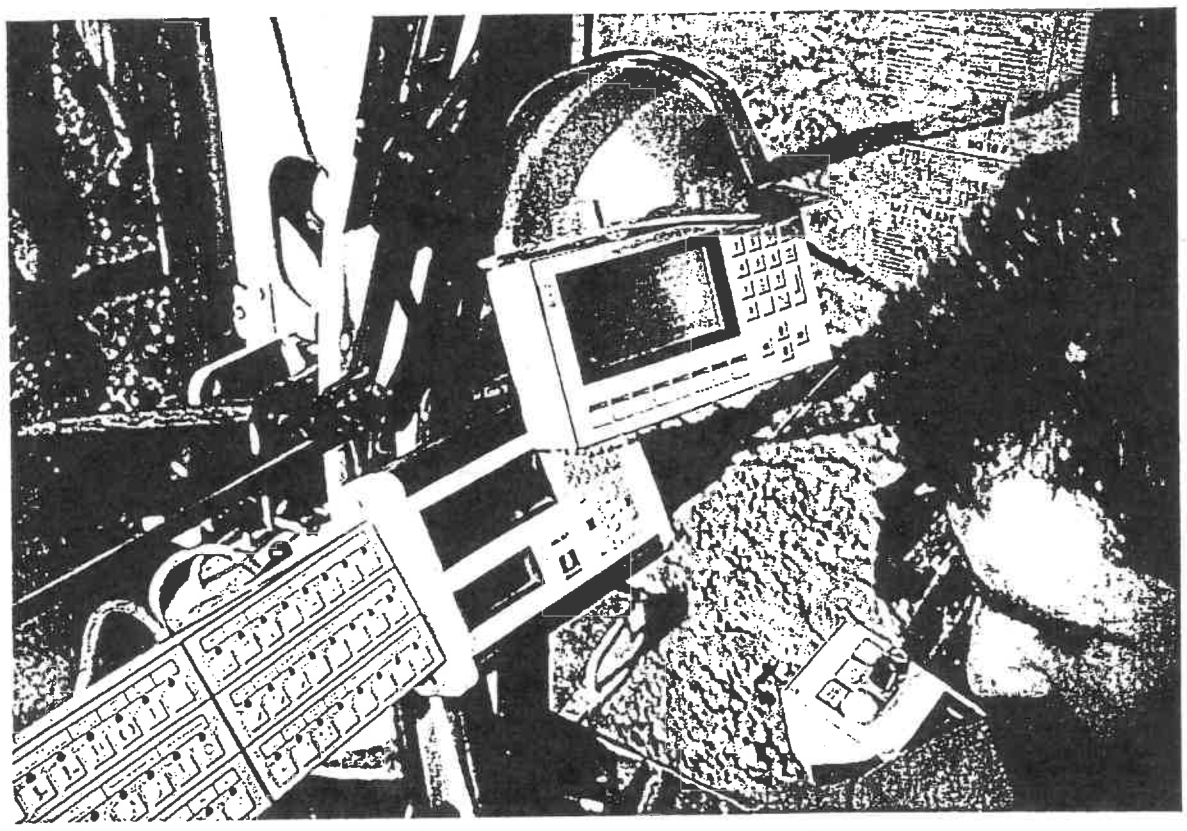
Drill rod dia 350 mm



typical print out of equipment process data



typical print out concrete pressure and deviation protocol



Quality Control - Material

- Material components and ready mixed concrete is regularily tested on the basis of local codes and a company´s quality plan (e.g. quality of cement, workability of fresh concrete (slump), strength of concrete (compressive strength after 28 days)

Quality Control – Construction Process

- Visualising and storing of relevant process data such as:
  -- depth
  – torque and crowd force
  – crowd speed
  -- deviation
  – concrete pressure and concrete volume

  Data are analysed and printed as production record and quality control sheets

- The maximum bearing capacity of a pile is limited by:

  Internal bearing capacity (allowable stress in concrete section)

  $V_{max} = A \times \beta_{all}$

  External bearing capacity (transfer of load into the soil in skin friction and base pressure)

  $V_{max} = A_{skin} \times \tau + A_{base} \times \sigma_{base\ pressure}$

Internal bearing capacity

- allowable concrete stress is dependent on grade of concrete and locally applicable material code.
- **Example DIN**

Grade of concrete	B 25
ultimate stress	β ult = 25 MN/m²
calculated stress	β calc = β ult x 0.7 = 17,5 MN/m²
factor of safety	2,1
σ all = 17,5 / 2,1 =	**8,3 MN/m²**

- **Example British Standard BS**

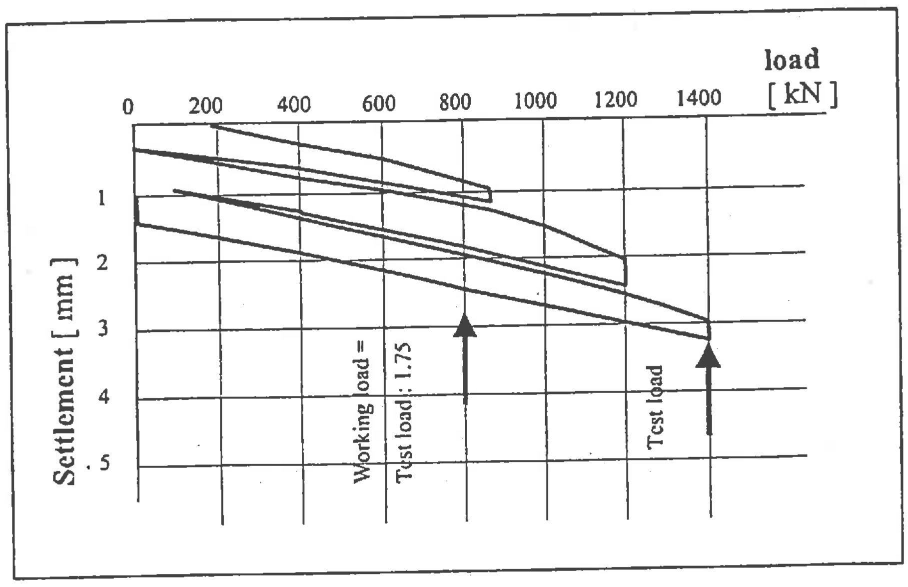
Grade of concrete	Grade 25
ultimate stress	β ult = 25 MN/m²
factor of safety	4
σ all = 25 / 4 =	**6,25 MN/m²**

- **Maximum bearing capacity** (for concrete B 25)
  assumption σ all = 6 – 7 MN/m²

diameter 400 mm	750 – 900 kN
diameter 500 mm	1.200 – 1.500 kN
diameter 600 mm	1.800 – 2.200 kN

Static Load Test

- Results of static load tests on test piles (if possible up to failure load) are the basis for an optimum design. Tests should be performed prior to finalising the design and prior to starting the work.
- Static load tests on working piles (with a typical test load of 1,5 x working load) are executed as quality control method



- **load – settlement diagram (typical)**

  pile diameter 360 mm / pile length 12 m

  soil: sand, medium dense (SPT values 15 – 20)

  skin area 13,6 m²
  skin friction (calculated) 1.400 kN / 13,6 m² = **100 kN/m²**

External bearing capacity

- The calculation method for external bearing capacity (load transfer into the soil, calculation of pile length) is dependent on locally applicable codes and specifications.
- Calculation method according DIN 4014
- a) Summarization of typical generalized soil profile

  b) Calculation of ultimate values for skin friction and base pressure. Reference values are listed in tables in DIN 4014 (dependent on type of soil and strength of soil).

  c) Increase of tabular values as allowance for solil displacement effect
  The amount of increase of code values is dependent on engineering judgment. Therefore it is **highly recommended** to verify the assumptions by performing load tests.

  d) Calculation of pile length with assumed bearing values
  aim: optimum pile length for maximum possible load
- EXAMPLE
- required working load 1.200 kN

  required diameter 500 mm (criterion inner bearing capacity)

  soil profile (bearing values acc. to DIN)

	layer thickness	ult. skin friction at settlement of:		ult. base pressure at settlement of:	
		2% of Diameter	10% of Diameter	2% of Diameter	10% of Diameter
Fill	2 m	--	--		
medium dense SAND (SPT 20)	8 m	80 kN/m²	80 kN/m²		
dense SAND (SPT 35)		120 kN/m²	120 kN/m²	700 kN/m²	2.000 kN/m²

- **Calculation of pile length**

Base pressure (in layer 3)
settlement at 2% Diameter 700 kN/m² => 137 kN
settlement at 10% Diameter 2.000 kN/m² => 390 kN

Skin friction in medium dense sand (layer 2)
at 10 mm Settlement 80 kN/m² => 1.000 kN

Skin friction in dense sand (layer 3)
at 10 mm Settlement 120 kN/m² => 188 kN / m pile

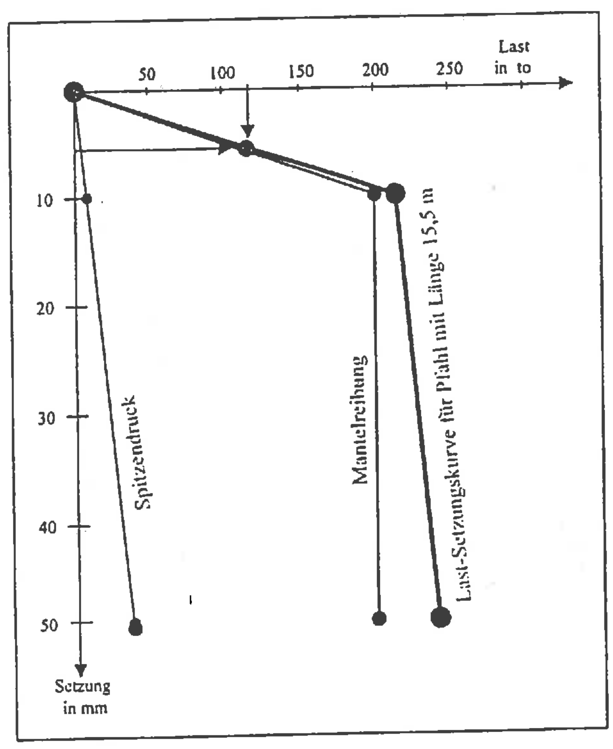
embeddment length in layer 3
ultimate load (1.200 kN x 2 (factor of safety)) 2.400 kN

2.400 kN	ultimate load
– 390 kN	base pressure
– 1.000 kN	skin friction (layer 2)
1.010 kN	remaining load (to be transferred in layer 3)

=> 1.010 kN / 188 kN per m => 5,5 m embeddment length

Total pile length

2 m	in fill
8 m	in layer 2
5,5 m	in layer 3
15,5 m	**total length**



Jobsite Reference

Project:

- foundation for supermarket, Debrecen (Hungary)
  Company: Jägerbau (Hungary)

Quantities:

- 1.300 no foundation piles (TYPE A), Diameter 360 mm
  Length 12 m, reinforcement length 5 m
  working load 85 to
- 6.300 no piles for soil improvement (TYPE B), Diameter 360 mm
  Length 7 m, not reinforced

Performance:

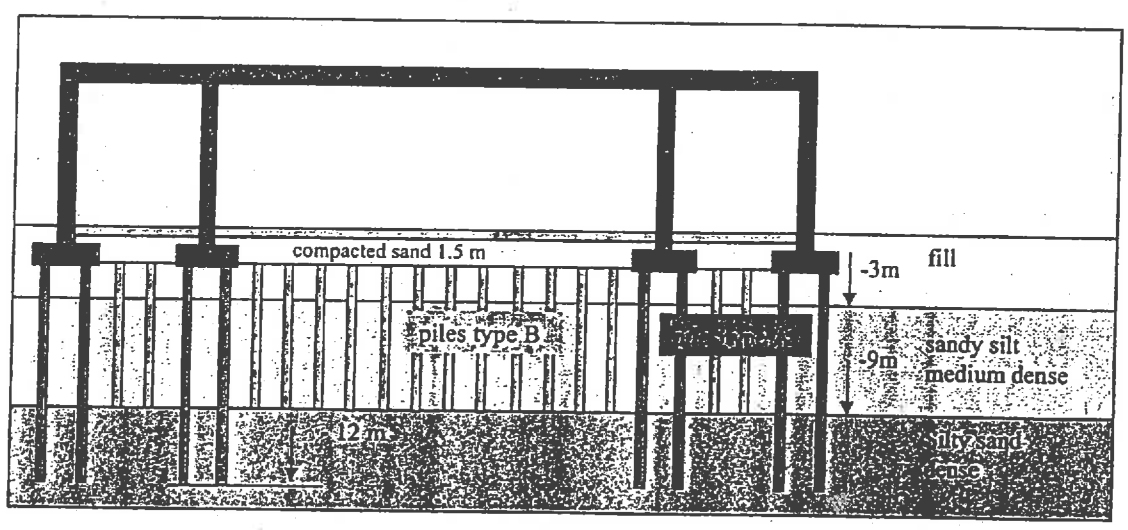
- TYPE A typical performance

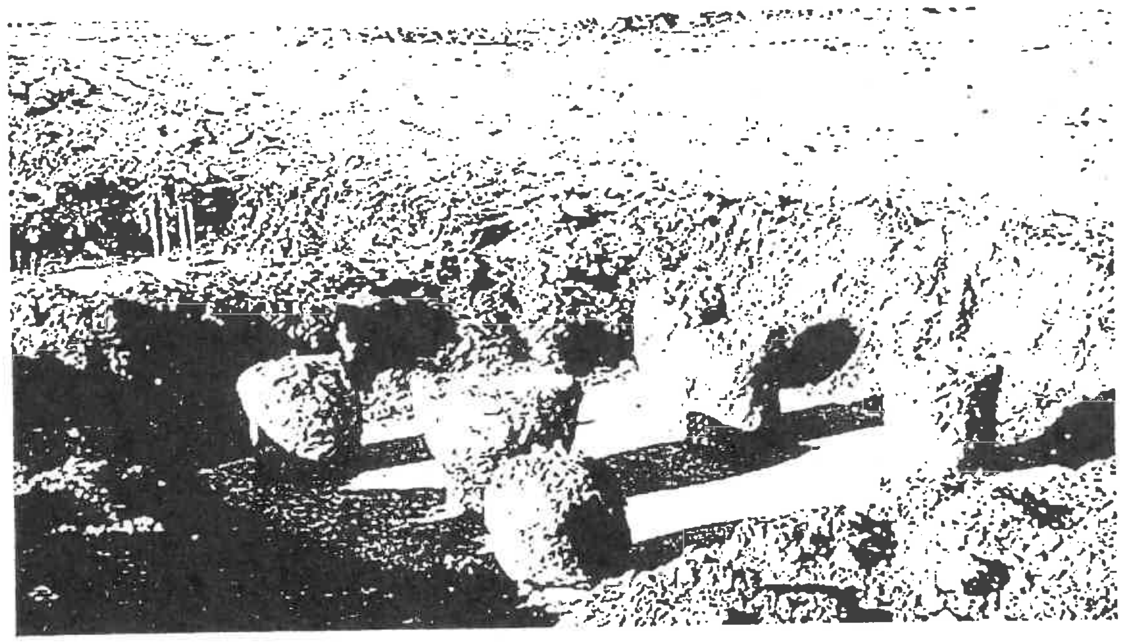
Positioning	5 min
Drilling 12 m	4 min
Concreting	4 min
total time 1 pile	**13 min**

- TYPE B typical performance

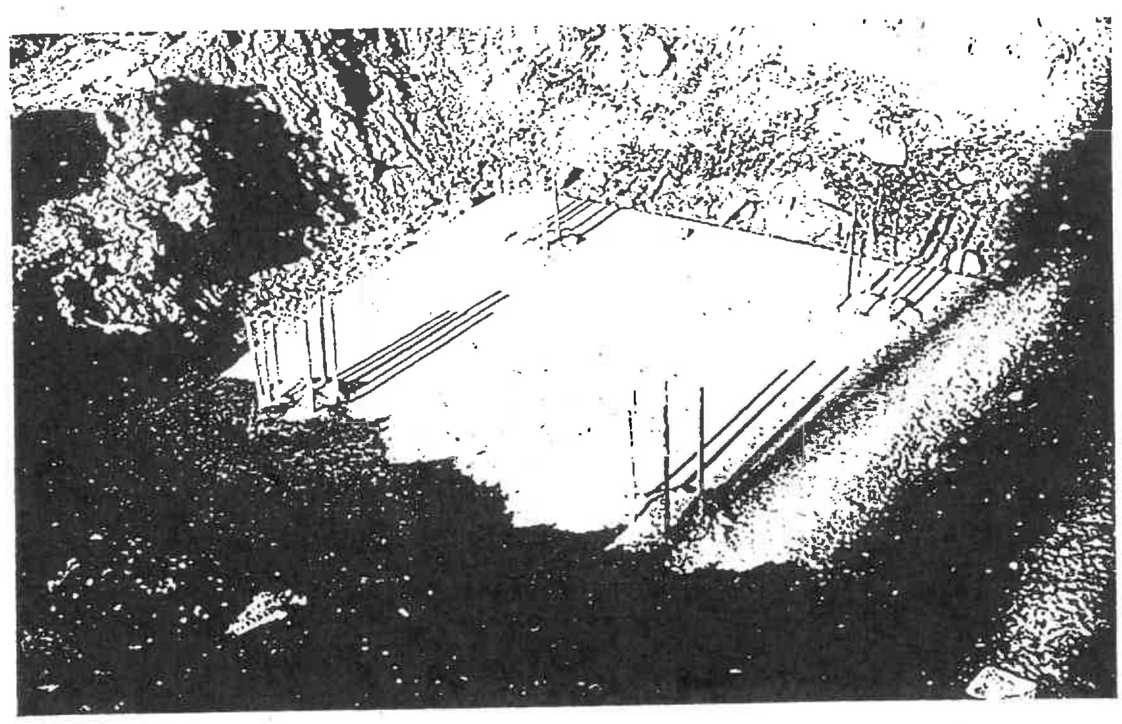
total time for 1 pile	**10 min**

- **construction period for 7.600 piles**
  2 months with 3 rigs





Pile group 6 piles (before trimming of pile heads)



pile group with 4 piles (after trimming of pile heads)

Auger-Displacement piles (ADP)

Post-tensioned mat

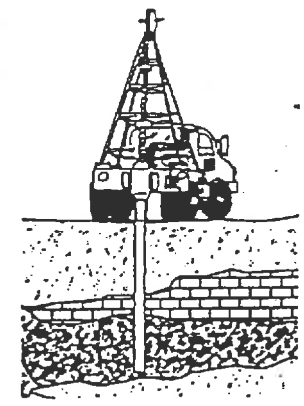
Use of Geotextiles for Foundation Improvement

STANDARD PENETRATION TEST

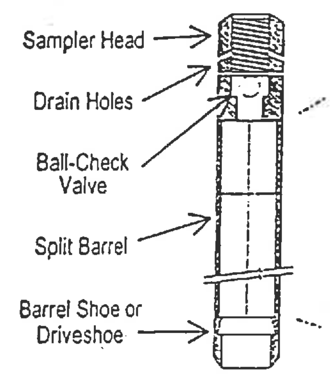
The Standard Penetration Test (SPT) is a widely used method of in-situ testing of soils and sampling. The test method has been now "Standardized" as "Penetration Test and Sampling of Soils" under the ASTM (American Society for the Testing of Materials) Designation D-1586. The test involves driving a 18 or 24 inch (30 centimeter) long, longitudinally split sampler with a 140 pound (63.5 Kg.) hammer falling freely through 30 inches (75 centimeter) drop. The number of blows required for each 6 inches of driving is recorded. The number of blows required for the last 12 inches (30 centimeter) of driving is recorded as the "N" value. The N Value from the SPT has been correlated to various soil engineering properties and allows a conservative estimate of the soil behavior under loading conditions.

The sampler is usually driven from the surface or from a depth of one foot below the surface, and is driven to provide continuous samples to a depth of about 6 to 10 feet below the surface. The borehole is then advanced by rotary drilling using either a hollow stem auger or rotary drilling equipment using drilling mud. When drilling with the bentonite mud is difficult, a casing is driven and the sampling continued inside the casing. The casing will be advanced as the boring progresses. After completion of the test boring, it is plugged, sometimes using a neat cement grout.

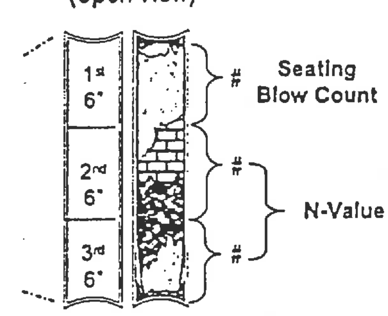
Representative samples obtained from the split spoon sampler are visually examined, classified according to the Unified Soil Classification System (ASTM D-2487) and packed and transported for further laboratory tests. These samples are considered "disturbed" samples and special techniques are used for obtaining relatively "undisturbed" samples for use in more elaborate laboratory tests. Samples are normally retained in the laboratory and discarded after 30 days unless otherwise advised by the client.



SPT Sampler

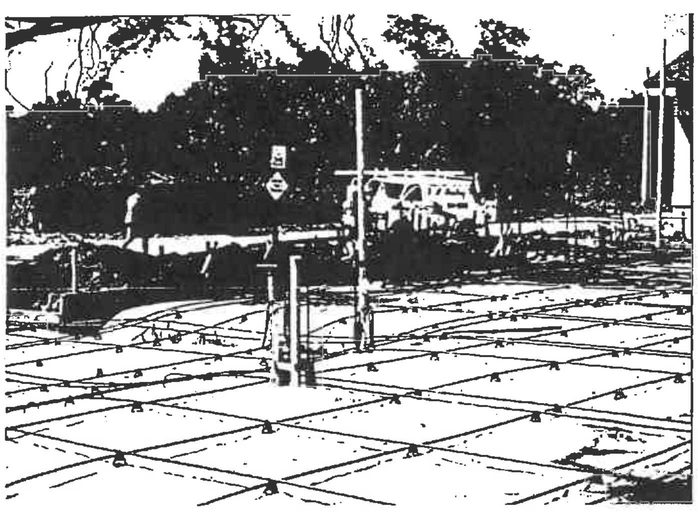


Sampler
(open view)



POST-TENSIONED MAT FOUNDATION

A mat foundation (or "raft" as it is called in the English-speaking world) is a flat concrete structural slab that distributes the load of the structure over its footprint. It might be considered a combined footing covering the entire loaded area. Mat foundations are broadly divided into two categories, rigid and flexible. Rigid mats are thick and rely on the concrete strength to resist the shear and moments. Flexible mats on the other hand are allowed to deflect in response to the loading conditions. However, as concrete is strong on compression but weak in tension, design of a flexible mat is constrained by concrete's modulus of rupture and flexural strength. If the concrete could be pre-compressed with a force that has to be overcome prior to relying on its own flexural strength, we have in effect augmented the tensile capacity. This is achieved by post-tensioning the mat. Post-tensioned mat uses high-strength 1860 MPa (270ksi) stranded wire stressed to 70 percent of its yield at final set. This applies compression to the concrete and thus optimizes the use of steel. More importantly we are getting this advantage before the concrete cracks, not after, as with conventional reinforcing.



Post-tensioned mats are particularly suited for areas with expansive clays and highly compressible soils. A post-tensioned mat is commonly used in residential buildings from single-family homes to low-rise apartment buildings. In the picture shown here for a 290 m^2 home in Florida, the cables were placed at 500 mm centers. This location was subject to sinkhole activity. Some of the advantages of post-tensioned mats are:

- Improved Modulus of Rupture resulting in a more structurally effective foundation.
- Deflection control — Mats could be designed to tolerate 70 mm differential settlement.
- Shrinkage crack control.
- Savings in reinforcement.
- Less Concrete.
- Fewer joints in slab – We have designed 20 m x 20 m slab without joints.
- Suitable for one to six-story high residential and commercial buildings.
- Savings in excavation, shoring and ease of preparing the subgrade.
- Faster construction clean up resulting in time savings.

Sources:
ASCE Civil Engineering
Post-tensioning Institute
Precast/Prestressed Concrete Institute

For more information call:
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P O Box 7525, Seminole, FL 33775
www.gate.net/~rkannan

USE OF GEOTEXTILE FOR FOUNDATION IMPROVEMENT

Ramanuja Chari Kannan, P.E., R. C. Kannan & Associates, Inc.
Unites States of America

ABSTRACT

Light commercial and industrial buildings in Florida are normally supported on spread footings. If an unsuitable soil condition such as organic matter or debris is encountered, piles or mat foundations are used. However, for low-rise buildings it is not economical and feasible. Removal and replacement of the unsuitable soil is as expensive as using a piled foundation system for small structures. However, by using a layer of geotextile, site preparation could be achieved at a reasonable cost.

In the two case histories reported in this paper, a layer of geotextile provided economical alternative to excessive excavation and backfilling. In one case organic material was encountered and in the other, construction debris was encountered. Use of geotextile reduced the amount of excavation required, reduced the amount of dewatering required and helped cut both the cost and time required to complete the site preparation work. The buildings have performed well. The use of geotextiles in cases such as these will provide both economic and structural benefits.

INTRODUCTION

Geotextiles have been used in pavements as a structural element for load distribution and soil separation. However in case of buildings, the use of geotextiles have not found a similar load distribution application. Foundation design for ground-supported buildings is based on bearing capacity and settlement calculation. Theoretical analysis is available for layered systems involving sandy and clayey soils. However, when highly compressible organic matter or highly non-homogenous debris is encountered, analysis becomes a complicated issue. In the case of organic soils, practically no theoretical analysis exists. In the case of non-homogenous materials such as construction debris, landfills, etc. in-situ load tests have been found more appropriate than theoretical analysis. Unlike foundation design for buildings, extensive use of geotextiles has produced empirical methods of analysis based on measured performance for the design of pavements, reinforced earth systems and slope stabilization. Theoretical analysis has also been made possible by the use of limit-equilibrium methods. Design procedures are simplified by combining the theoretical concepts and experience gained from full-scale field performance tests.

Geotextiles have been successfully used in road construction, slope stabilization and in earth-reinforced retaining walls. The methods of design suggested are based on performance tests. In foundation design however, the use of geotextiles has not gained wide acceptance. One of the reasons for this is the fact that full-scale load tests have not been feasible in the case of buildings. The other factor is that scale effects that are applicable to roads may not be applicable to buildings. This paper lists two case histories where geotextiles have been successfully used to provide adequate foundation support in difficult site conditions.

In the two case histories described in this report, geotextiles were used primarily as a structural element to aid in load distribution. A geotextile layer was introduced as a stratum of high-tensile strength material that will help reduce the stresses and therefore the deflections in the underlying strata. Yamauchi and Kitamori (1985) recommended a method of analysis to estimate the bearing capacity of such foundation systems. We also used classical bearing capacity analysis (Terzaghi, 1943) to estimate the bearing capacity of the prepared foundation soils and Schmertmann's method (1970) for settlement analysis. In both methods of bearing capacity estimation, the critical factor is the thickness of sand layer over the fabric. We therefore designed the thickness of the sand layer such that the stress increment at the depth where the fabric was placed was less than 10% of the loading intensity of the footings. In one of the two cases described, a single-story commercial building was constructed on a site where peat was encountered at a shallow depth below the foundation. The unknown behavior of organic soils and the high water table conditions were the limiting factors in this design. In the second case, construction debris was found. Using a layer of geotextile helped prevent excessive removal of debris that had been dumped perhaps 30 years ago. The unknown nature, depth and extent of buried debris presented a problem, which was essentially overcome by using a layer of geotextile to provide separation and load distribution for two-story residential buildings. The geotextile fabric was also used in both cases to separate the unsuitable soils from the foundation soils.

In the two cases discussed, the structures are light and the use of other methods of deep foundations to circumvent the unsuitable soils were expensive. The use of a woven high density polyethylene fabric was the best solution in both cases. In structures of this type in sand, settlement problems normally appear as step cracks within one to six months of completion. In the cases described here, no cracks have appeared for over four years. The use of geotextiles in foundation design is therefore open for further study, analysis and wider use. There may also be other applications such as reducing the swell potential, which should be investigated.

CASE HISTORY ONE: FOUNDATIONS OVER ORGANIC SOILS

The maintenance building for Eckerd College in St. Petersburg, Florida was proposed to be located on a site close to the waters of Tampa Bay and adjacent to a large pond. The building was a single-story structure, with a concrete masonry block office building attached to a steel-framed workshop building. The loading intensity from the load bearing walls was not anticipated to exceed 4 kN/m. Subsurface exploration conducted before the construction began showed a layer of organic soils. These soils ranged from sand mixed with fibrous organic matter to a layer of peat at three of the five test pits dug within the building footprint. The organic material was located at a depth between of 1.2 m to 1.8 m below the ground level. The thickness of the organic material ranged from about 25 mm to 600 mm. This was a peat-like material mixed with sand and the organic content determined by ASTM D2974-87 test method was 18 to 23%. The soil profile used in the engineering analysis is shown in the table below.

Table 1. Soil Properties of subsurface soils

Depth From (m)	Depth to (m)	Soil Description	Dry unit weight γ_d in kN/m^3	Estimated friction angle $\Phi_{effective}$ degrees
Ground	1 to 1.2	Poorly graded SAND (SP) and Fill materials	15.5 + 0.3	23 to 27
1.2	1.8	Muck and Peat (Pt) {Encountered to 3 m depth at some locations}	13.5 + 0.4	Not applicable
1.8	2.3	Poorly graded SAND (SP), Dark brown	15.8 + 0.4	22 to 25
2.3	5.2	Poorly graded SAND (SP), gray	15.8 + 0.3	22 to 25
5.2	6.1	Poorly graded SAND with SILT (SP-SM)	15.5 + 0.3	23 to 26

Prior to construction, it was decided that the organic soils should be completely removed where they were encountered and replaced with clean granular soils. When construction began, it was noticed that the muck and peat were present under the entire footprint of the building. The water table was at 1.2 m below the surface and the organic matter was encountered just below the water table. As the original intention was to remove the organic soils, dewatering the excavation was attempted. However, unexpected heavy rains and the proximity of the large bodies of water made dewatering practically impossible. With difficulty and working between drier days, the contractor was able to excavate to a depth of about 2.4 m. Without a cut-off wall or the use of a sheet pile wall the dewatering and excavation could not be continued. Besides, there was no assurance that the adjacent detention ponds could be maintained during further dewatering.

After discussing and evaluating other possible alternatives such as deep foundations, two options that were economically feasible were left. The first option was to redesign the building to be supported on a post-tensioned mat and the second alternative was to use a layer of geotextile as stabilizing layer. A number of factors contributed in favor of trying geotextile, which included the light structural loads, the need to complete the building on time, the delays associated with the re-design, bidding and finding a specialty contractor with post-tensioning experience and of course, the cost escalation. The solution recommended was as follows.

With the excavation already 2.4 m deep and more rains expected, the first priority was to backfill to an elevation above the water table. We recommended that at least 600 mm of granular fill be placed at the bottom of the excavation and compacted with a vibratory tamper. A layer of geotextile was then to be laid on the compacted surface. The geotextile recommended was a woven high-density polyethylene 0.75 mm thick. The minimum material properties specified were 25.4 x 28.0 kN/mWide Width Tensile Strength (ASTM D4595), Grab Tensile Strength of 1110 N, Mullen Burst Strength of 3445 kPa. Elongation was not specified, as it was estimated to be nominal (under 5%). Granular fill was then to be placed in 300 mm thick lifts and compacted to a minimum of 98% of the modified Proctor maximum density (AASHTO T-180.) The total thickness of the fill from the fabric layer to the finished floor elevation was about 2.4 m. The bottom of the footings was 600 mm below the finished floor and hence the footings were placed at or near the existing ground surface. With this arrangement, the stresses at the fabric elevation were estimated to be less than 0.4 kN/m^2 and hence the underlying organic soils are stressed to lesser levels. As the organic soils are always expected to be below the water table, no significant volume changes were expected. Estimated settlements for the structure were 15 to 20 mm.

Since the structure was completed in early 1998, it has performed well. Though settlement has not been measured, the building was observed at six-month intervals for two years and no settlement cracks have been noticed. As the workshop part of the building is steel-framed structure, it was considered feasible that minor settlements could be adjusted. But that has not been necessary.

CASE HISTORY TWO: FOUNDATIONS OVER DEBRIS

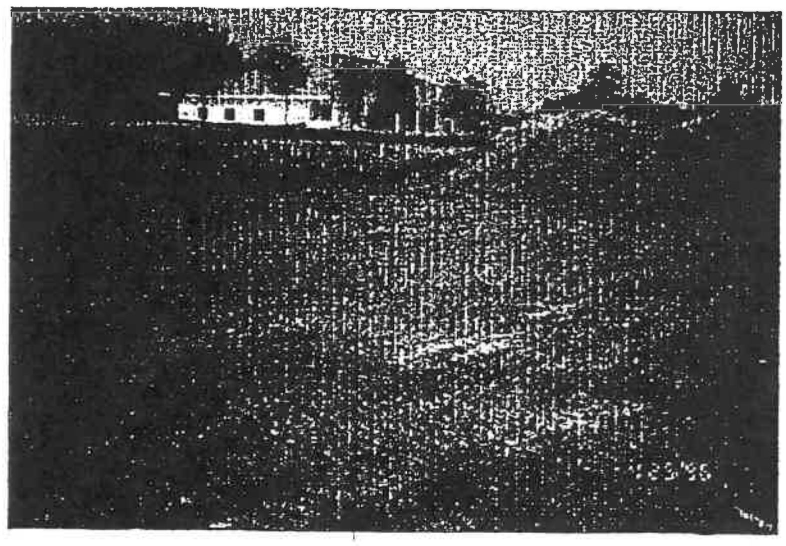
Sun City Center is a retirement community in Hillsborough County, Florida, about 40 km south of Tampa. Development began in the late sixties and has continued since then. When the earliest developers arrived, there were no facilities other than a river located about 10 km south of the area. The site was primarily agricultural land and a watershed for the Little Manatee River. Our client, who is developer of retirement communities, began developing a 47-hectare parcel in 1988. The site was believed to be virgin land except for a 4-hectare parcel that was used in the seventies as staging area for the construction of other housing developments in the vicinity. The initial phases of construction, which included a golf course, went on smoothly for the first three years. When development began on one of the sub-parcels adjacent to an existing housing in 1990, we discovered buried debris, consisting mostly of construction debris within an area 200 m wide, 100 m long and extending from 1 to about 5 m deep. The surface cover was barely 300 mm thick and the debris included such items like water heaters, piles cut off, concrete washout from trucks, pieces of lumber, plastic wraps, steel cables and reinforcement bars to name a few. Obviously one of the earlier developers had discovered an ideal source of fill and a landfill very conveniently located. Fortunately,. the debris was non-toxic, but its removal was not economically feasible. Two story apartment buildings were slated for construction in this area. Driving piles was impractical because of the nature of the debris.

Based an elastic settlement analysis (Schmertmann, 1970), it was decided that the buildings could be supported on spread footings provided that a uniform, compacted stratum at least 2 m thick could be created below the foundation elevation. It was decided therefore that a partial removal of the debris would be attempted and a layer of geotextile will be used to separate the debris-laden soil from the prepared foundation soils. The subsurface soil conditions showed that excellent fill material will be available from the excavation of detention ponds at the site and the excavated debris could be disposed at site. This was the most economical solution.

The use of geotextile in this case permitted the design of shallow spread footings for the buildings. The only other alternative was to abandon the site and move the buildings elsewhere; an option that could not be exercised once the construction of the golf course had begun. The geotextile recommended in this case was a woven high density polyethylene 1 mm fabric. The minimum material properties specified were 30 x 35 kN/mWide Width Tensile Strength (ASTM D4595); Grab Tensile Strength of 1400 x 1400 N; Mullen Burst Strength of 4480 kPa. The geotextile was required to meet AASHTP M288-96 requirements for Class 2 Separation and Class 1 Stabilization. The site was cleared and the debris was excavated to a depth of about 2.4 m below he existing ground surface. The finished floor elevation was about 600 mm above the existing ground surface. Though the composition of the debris varied, large pieces of concrete, timber, etc were found to be less than 20% of the volume of the earth excavated. Most of the debris was between gravel and boulder size, rarely exceeding 500 mm in any dimension. As the debris was excavated, it was sorted to separate usable soil from the debris. To obtain a level surface to lay the fabric, the voids in the debris laden soil were filled with crushed concrete aggregate, a recycled material that has been used as a road base for over a decade in this area. This gave a working surface that could be rolled with a heavy vibratory compactor and gave a surface to lay the fabric. The prepared surface is shown in the frame below.

Once the fabric was laid, clean granular fill was placed in 300 mm lifts and compacted to a minimum density of 98% of the modified Proctor maximum density. The compacted surface was designed to provide an allowable bearing capacity of 150 kN/m^2.

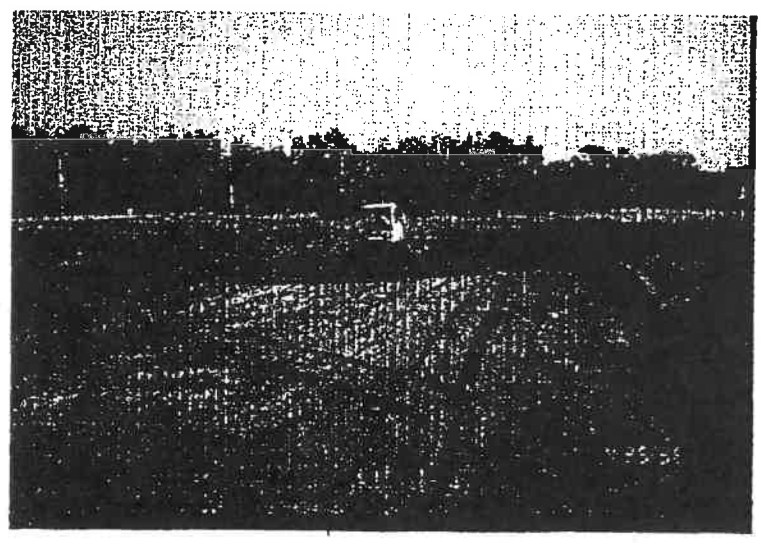
Figure One: Prepared surface over buried debris



At the time of this writing, seven buildings and the roads have been completed. Construction on the other six buildings and a club house is scheduled to begin in last quarter of 2000.

In addition to the buildings, some of the utilities and pavements located in the area of the debris were also treated in a similar manner. A layer of woven high-density polyethylene was placed about 600 mm below the bottom of the pipes before the pipe bedding was prepared. In case of the roads, the fabric was laid at a higher elevation, below the base course and integrated with the subgrade.

Figure Two: Partially completed building pad.



OTHER APLICATIONS

In recent years we have come across numerous cases of single-story homes damaged by sinkhole activity all over the Tampa Bay area. A sudden dramatic collapse occurs infrequently, but excessive damage due to ground subsidence caused by solution activity is common. Settlement in the range of 15 to 30 mm is often encountered. In trying to remedy many of these structures this writer has recommended (Kannan, 1999) that the residential structure be supported on post-tensioned mat, placed on stabilized ground. After grouting the voids in the limestone, we have recommended the use of geotextile to provide a uniform bearing stratum. Geotextile in this case also aided in retarding the migration of foundation soils to subsurface strata due to solution activity.

CONCLUSION

These case histories have established the feasibility of building light structures on compacted fill placed over geotextile. A theoretical method of design has not been developed, but in these two cases conventional bearing capacity analysis was used. This innovative solution saved both time and money involved in extensive excavation, dewatering and bringing imported fill.

Future research should focus on conducting full-scale load tests where settlements could be measured. Research should also focus on developing a theory for load-distribution when a layered system includes a negligibly thin layer as compared to the thickness of other soil strata. We have also found that the modulus of subgrade reaction for post-tensioned mat design can be substantially improved by using a layer of geotextile. This also needs further study and research in developing design guidelines.

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