August 27, 2013
Revised October 18, 2013
File No. 20572

MGA North, LLC
16300 Roscoe Boulevard
Van Nuys, California 91406

Attention: Leon Benraimon

Subject: Geotechnical Engineering Investigation
Proposed Mixed-Use Development
20000 Prairie Street, Chatsworth, California

Dear Mr. Benraimon:

This letter transmits the Geotechnical Engineering Investigation for the subject site prepared by Geotechnologies, Inc. This report provides geotechnical recommendations for the development of the site, including earthwork, seismic design, retaining walls, excavations and foundation design. Engineering for the proposed project should not begin until approval of the geotechnical investigation is granted by the local building official. Significant changes in the geotechnical recommendations may result due to the building department review process.

The validity of the recommendations presented herein is dependent upon review of the geotechnical aspects of the project during construction by this firm. The subsurface conditions described herein have been projected from limited subsurface exploration and laboratory testing. The exploration and testing presented in this report should in no way be construed to reflect any variations which may occur between the exploration locations or which may result from changes in subsurface conditions.

Should you have any questions please contact this office.

Respectfully submitted,
GEOTECHNOLOGIES, INC.

REGISTERED PROFESSIONAL ENGINEER
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Exp. 6/30/14
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GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT
20000 PRAIRIE STREET
CHATSWORTH, CALIFORNIA

INTRODUCTION

This report presents the results of the geotechnical engineering investigation performed on the subject site. The purpose of this investigation was to identify the distribution and engineering properties of the geologic materials underlying the site, and to provide geotechnical recommendations for the design of the proposed development.

This investigation included 21 exploratory excavations, collection of representative samples, laboratory testing, engineering analysis, review of published geologic data, review of available geotechnical engineering information and the preparation of this report. The exploratory excavation locations are shown on the enclosed Plot Plan. The results of the exploration and the laboratory testing are presented in the Appendix of this report.

PROPOSED DEVELOPMENT

Information concerning the proposed development was furnished by the client. The proposed project consists of the construction of three mixed-use structures. The proposed structures will be up to five stories in height. Some of the structures will be built near the existing site grade others will be serviced by one level of subterranean parking. Column loads are estimated to be between 400 and 900 kips. Wall loads are estimated to be between 6 and 12 kips per lineal foot. Grading for the proposed at-grade structures is expected to consist of removal and recompaction of existing unsuitable soils for the construction of uniform building pads. Grading for the proposed structures which will be serviced by below-grade parking will consist of excavations up to 15 feet.

In addition to the proposed mixed-use structures, a vehicular bridge and a pedestrian bridge are proposed to be built across the existing flood control channel. The location of the proposed structures is shown in the enclosed Plot Plan.

Any changes in the design of the project or location of any structure, as outlined in this report, should be reviewed by this office. The recommendations contained in this report should not be considered valid until reviewed and modified or reaffirmed, in writing, subsequent to such review.

SITE CONDITIONS

The site is located at 20000 Prairie Street in Chatsworth, California. It is estimated that the site is between 20 and 25 acres in area. The site is bounded by Prairie Street to the north, railroad tracks, followed by an office complex to the east, railroad tracks to the south, and Winnetka Avenue to the west. The site is shown relative to nearby topographic features on the enclosed Vicinity Map.

A City of Los Angeles flood control channel runs along the southern and western property lines. The location of this channel is shown in the enclosed Plot Plan. The channel is concrete lined and approximately 20 feet wide along the western property line, and 30 feet wide along the southern property line. The depth of the channel was measured to range between 8 and 9 feet.

A topographic survey was not furnished to this firm for the preparation of this report. According to the USGS Topographic Map for the Canoga Park 7½ minute Quadrangle, site elevations vary from 860 feet at the northwestern corner, to 848 feet at the southeastern corner. The site slopes gently to the southeast at an average 120 to 1 (H to V) gradient.

The site is currently developed with the former L.A. Times printing plant, and several asphalt-paved parking lots. The location of the printing plant structure is shown in the enclosed Plot Plan. It is the understanding of this firm that this existing structure consists of a single tall-story, built near the existing site grade. It is anticipated that this structure will remain, and will be incorporated to the proposed development. It is the understanding of this firm that geotechnical recommendations to aid in the re-adaptation of the existing structure are not needed at this time.

Vegetation on the site consists of grass lawns, mature trees, bushes and shrubs. Drainage across the site appears to be by sheetflow to the flood control channel to the south and west.

PREVIOUS SITE WORK

Geotechnologies, Inc., May 28, 2008, Geotechnical Engineering Investigation, Proposed Renovation of Existing Commercial Structure, File No. 19668.

This firm performed a Geotechnical Engineering Investigation on a portion of the subject site. This investigation pertained to the renovation of the existing structure. Five borings and two test pits were excavated to depths of 6 and 50 feet below the ground surface. The logs of these excavations are included in the Appendix of this investigation, and their locations are shown on the attached Plot Plan. The laboratory testing from this previous investigation is also incorporated onto the enclosed Plate B-1.

Percolation testing was conducted in Test Pit 1 as part of the investigation. The location of Test Pit 1 is shown in the enclosed Plot Plan. The results have been incorporated into this investigation, and are presented in the "Stormwater Disposal" Section of this report.

GEOTECHNICAL EXPLORATION

FIELD EXPLORATION

The site was explored on July 8, 9, and 10, 2013 by excavating 21 borings. The borings were drilled to depths between 20 and 50 feet below the site grade with the aid of a truck-mounted drilling machine using 8-inch diameter hollowstem augers. The exploration locations are shown on the Plot Plan and the geologic materials encountered are logged on Plates A-1 through A-21.

Results from a previous exploration of the site, conducted by this firm in 2008, have been incorporated into this current investigation. As part of the previous exploration, five borings and two test pits were excavated to depths ranging between 6 and 50 feet below the site grade. The borings were drilled with the aid of a truck-mounted drilling machine using 8-inch diameter hollowstem augers, while the test pits were excavated with the aid of hand tools. The logs of these explorations are included in the Appendix of this investigation, and their location is shown on the attached Plot Plan.

The location of exploratory excavations was determined from hardscape features shown on the attached Plot Plan. The location of the exploratory excavations should be considered accurate only to the degree implied by the method used.

Geologic Materials

Fill materials were encountered during exploration to depths ranging between 1 and 12½ feet below the existing site grade. The deepest fill was encountered in Boring 12. Most of the site is underlain by only 2½ feet of fill. The fill consist of a mixture of sand, silt, and occasional clay. The fill ranges between yellowish brown and dark brown in color, and is moist, medium dense to dense, or stiff, and fine grained with occasional gravel.

The fill is in turn underlain by alluvial soils consisting of interlayered mixtures of sand, silt and clay. The alluvial soils range from yellowish brown to dark brown in color, and are slightly moist to moist, stiff to very stiff, or medium dense to very dense, and fine to coarse grained, with occasional gravel, cobbles, and caliche cementation. More detailed descriptions of the geologic materials encountered may be obtained from individual logs of the subsurface excavations.

Groundwater

Groundwater was not encountered during exploration, conducted to a maximum depth of 50 feet below the existing site grade. The historically highest groundwater level was determined by review of the Canoga Park 7½ Minute Quadrangle Seismic Hazard Evaluation Report, Plate 1.2, Historically Highest Ground Water Contours (CDMG, 2005). Review of this plate indicates that the historically highest groundwater level on the site ranges from 41 feet below grade at the southeastern corner, to 52 feet below grade at the northwestern corner. A copy of this plate is included in the Appendix as Historically Highest Groundwater Levels Map. For the purpose of this geotechnical analysis, a historically highest groundwater level of 40 feet has been assumed for the site.

Fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, and other factors not evident at the time of the measurements reported herein. Fluctuations also may occur across the site. High groundwater levels can result in changed conditions.

Caving

Caving could not be directly observed during excavation of the borings due to the continuously-cased design of the hollowstem augers. Caving was not experienced during excavation of the test pits. However, based on the experience of this firm, large diameter excavations that encounter granular, cohesionless soils will most likely experience caving.

SEISMIC EVALUATION

REGIONAL GEOLOGIC SETTING

The subject property is located in the Transverse Ranges Geomorphic Province. The Transverse Ranges are characterized by roughly east-west trending mountains and the northern and southern boundaries are formed by reverse fault scarps. The convergent deformational features of the Transverse Ranges are a result of north-south shortening due to plate tectonics. This has resulted in local folding and uplift of the mountains along with the propagation of thrust faults (including blind thrusts). The intervening valleys have been filled with sediments derived from the bordering mountains.

REGIONAL FAULTING

Based on criteria established by the California Division of Mines and Geology (CDMG) now called California Geologic Survey (CGS), faults may be categorized as active, potentially active, or inactive. Active faults are those which show evidence of surface displacement within the last 11,000 years (Holocene-age). Potentially-active faults are those that show evidence of most recent surface displacement within the last 1.6 million years (Quaternary-age). Faults showing no evidence of surface displacement within the last 1.6 million years are considered inactive for most purposes, with the exception of design of some critical structures.

Buried thrust faults are faults without a surface expression but are a significant source of seismic activity. They are typically broadly defined based on the analysis of seismic wave recordings of hundreds of small and large earthquakes in the southern California area. Due to the buried nature of these thrust faults, their existence is usually not known until they produce an earthquake. The risk for surface rupture potential of these buried thrust faults is inferred to be low (Leighton, 1990). However, the seismic risk of these buried structures in terms of recurrence and maximum potential magnitude is not well established. Therefore, the potential for surface rupture on these surface-verging splays at magnitudes higher than 6.0 cannot be precluded.

SEISMIC HAZARDS AND DESIGN CONSIDERATIONS

The primary geologic hazard at the site is moderate to strong ground motion (acceleration) caused by an earthquake on any of the local or regional faults. The potential for other earthquake-induced hazards was also evaluated including surface rupture, liquefaction, dynamic settlement, inundation and landsliding.

Surface Rupture

In 1972, the Alquist-Priolo Special Studies Zones Act (now known as the Alquist-Priolo Earthquake Fault Zoning Act) was passed into law. The Act defines “active” and “potentially active” faults utilizing the same aging criteria as that used by California Geological Survey (CGS). However, established state policy has been to zone only those faults which have direct evidence of movement within the last 11,000 years. It is this recency of fault movement that the CGS considers as a characteristic for faults that have a relatively high potential for ground rupture in the future.

CGS policy is to delineate a boundary from 200 to 500 feet wide on each side of the known fault trace based on the location precision, the complexity, or the regional significance of the fault. If a site lies within an Earthquake Fault Zone, a geologic fault rupture investigation must be performed that demonstrates that the proposed building site is not threatened by surface displacement from the fault before development permits may be issued.

Ground rupture is defined as surface displacement which occurs along the surface trace of the causative fault during an earthquake. Based on research of available literature and results of site reconnaissance, no known active or potentially active faults underlie the subject site. In addition, the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. Based on these considerations, the potential for surface ground rupture at the subject site is considered low.

2010 California Building Code Seismic Parameters

Based on information derived from the subsurface investigation, the subject site is classified as Site Class D, which corresponds to a "Stiff Soil" Profile, according to Table 1613.5.2 of the 2010 California Building Code. This information and the site coordinates were input into the USGS Ground Motion Parameter Calculator (Version 5.1.0) to calculate the Maximum Considered Earthquake (MCE) Ground Motions for the site. The Maximum Considered Earthquake Ground motions are equivalent to the 2475-year recurrence interval ground motions adjusted by a deterministic limit. These values are consistent with the 2009 International Building Code requirements. Ground motion parameters for both the 2010 CBC (ASCE 7-05), and 2013 CBC (ASCE 7-10) are presented below.

2010 CALIFORNIA BUILDING CODE SEISMIC PARAMETERS	
Site Class	D
Mapped Spectral Acceleration at Short Periods (S_S)	1.786g
Site Coefficient (F_a)	1.0
Maximum Considered Earthquake Spectral Response for Short Periods (S_{MS})	1.786g
Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	1.191g
Mapped Spectral Acceleration at One-Second Period (S_1)	0.650g
Site Coefficient (F_v)	1.5
Maximum Considered Earthquake Spectral Response for One-Second Period (S_{M1})	0.975g
Five-Percent Damped Design Spectral Response Acceleration for One-Second Period (S_{D1})	0.650g

2013 CALIFORNIA BUILDING CODE SEISMIC PARAMETERS	
Site Class	D
Mapped Spectral Acceleration at Short Periods (S_S)	2.148g
Site Coefficient (F_a)	1.0
Maximum Considered Earthquake Spectral Response for Short Periods (S_{MS})	2.148g
Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	1.432g
Mapped Spectral Acceleration at One-Second Period (S_1)	0.684g
Site Coefficient (F_v)	1.5
Maximum Considered Earthquake Spectral Response for One-Second Period (S_{M1})	1.026g
Five-Percent Damped Design Spectral Response Acceleration for One-Second Period (S_{D1})	0.684g

The peak ground acceleration (PGA) and modal magnitude were also obtained from the USGS Probabilistic Seismic Hazard Deaggregation program (USGS, 2008). The results are based on a 10 percent in 50 years ground motion (475 year return period). A published shear wave velocity of 230 meters per second was utilized for Vs30 (Tinsley and Fumal, 1985). The deaggregation program indicates a PGA of 0.48g and a modal magnitude of 6.6 for the site.

Liquefaction

Liquefaction is a phenomenon in which saturated silty to cohesionless soils below the groundwater table are subject to a temporary loss of strength due to the buildup of excess pore pressure during cyclic loading conditions such as those induced by an earthquake. Liquefaction-related effects include loss of bearing strength, amplified ground oscillations, lateral spreading, and flow failures.

The Seismic Hazard Map for the Canoga Park Quadrangle by the State of California (CDMG, 1998), does not classify the site as part of a "Liquefiable" area. This determination is based on historic groundwater depth records, soil type, and distance to a fault capable of producing a substantial earthquake. A copy of this map is included in the Appendix.

Two site-specific liquefaction analyses were performed following the Recommended Procedures for Implementation of the California Geologic Survey Special Publication 117A, Guidelines for Analyzing and Mitigating Seismic Hazards in California (CGS, 2008). The enclosed liquefaction analyses were performed using the spreadsheet template LIQ2_30.WQ1 developed by Thomas F. Blake (Blake, 1996). This program utilizes the 1996 NCEER method of analysis. This semi-empirical method is based on a correlation between measured values of Standard Penetration Test (SPT) resistance and field performance data.

Groundwater was encountered not encountered during exploration, conducted to a maximum depth of 50 feet below the existing site grade. According to the Seismic Hazard Zone Report of the Canoga Park 7½-Minute Quadrangle (CDMG, 2005), the historical highest groundwater level for the site ranged between 40 and 52 feet below the existing ground surface. A historical highest groundwater level of 40 feet below the ground surface was conservatively utilized for the enclosed liquefaction analyses.

A moment magnitude (M_W) of 6.6 is utilized in the analysis, based on the USGS Probabilistic Seismic Hazard Deaggregation program (USGS, 2008). A peak ground acceleration of 0.48g is used in the enclosed liquefaction analyses. This value is the higher of the site specific peak ground acceleration associated with a 10 percent probability of being exceeded in 50 years for an alluvial site condition in this area of Los Angeles, obtained from the USGS Probabilistic Seismic Hazard Deaggregation program, and the peak ground acceleration based on the Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS}) divided by 2.5, in accordance with the California Building Code.

The enclosed "Empirical Estimation of Liquefaction Potential" analyses are based on the results obtained from Borings 6 and 19. Standard Penetration Test (SPT) data were collected at 5-foot intervals with a 140-lb automatic hammer. Samples of the collected materials were conveyed to the laboratory for testing and analysis. The percent passing a Number 200 sieve, and the Plasticity Index, (as determined by Atterberg Limits testing) of a fine grained soil layer encountered between depths of 40 and 50 feet are presented on the enclosed Plates E and F. Based on the enclosed liquefaction analyses, the lowest factor of safety calculated for soil layers considered susceptible to the occurrence of liquefaction is 1.65. Based on CGS Special Publication 117A (CDMG, 2008), a factor of safety against the occurrence of liquefaction greater than about 1.3 can be considered an acceptable level of risk where high-quality, site-specific penetration resistance and geotechnical laboratory data is collected. Based on the adjusted blow count data, results of laboratory testing, and the calculated factor of safety against the occurrence of liquefaction, it is the opinion of this firm that the potential for liquefaction at the site during the design earthquake is considered to be remote.

Dynamic Dry Settlement

Seismically-induced settlement or compaction of dry or moist, cohesionless soils can be an effect related to earthquake ground motion. Such settlements are typically most damaging when the settlements are differential in nature across the length of structures.

Seismic dry sand settlements were calculated utilizing Tokimatsu and Seed's procedure for the soils encountered in Borings 6 and 19 (Tokimatsu and Seed, 1987). Similar to the enclosed liquefaction analysis, a Magnitude Scaling Factor (M_w) of 6.6, and a peak horizontal acceleration of 0.48g were utilized for the enclosed dynamic dry settlement calculations. The potential for seismic dry settlement was evaluated to a depth of 50 feet. Based on these parameters, the enclosed seismically-induced dry sand settlement calculations resulted in a total settlement of 0.85 inches for Boring 6, and 0.49 inches for Boring 19.

Where at least two borings have been drilled to a depth of 50 feet, the City of Los Angeles, Department of Building and Safety requires that the differential seismically induced settlement be taken as no less than ½ of the total calculated settlement. Based on this requirement, the anticipated differential seismic settlement is expected to be 0.43 inches for Boring 6, and 0.25 inches for Boring 19. As a conservative measure, the total and differential settlements calculated for Boring 6 should be considered in the design of the proposed structures.

Tsunamis, Seiches and Flooding

Tsunamis are large ocean waves generated by sudden water displacement caused by a submarine earthquake, landslide, or volcanic eruption. Review of the County of Los Angeles Flood and Inundation Hazards Map, Leighton (1990), indicates the site does not lie within the mapped tsunami inundation boundaries.

Seiches are oscillations generated in enclosed bodies of water which can be caused by ground shaking associated with an earthquake. Review of the County of Los Angeles Flood and Inundation Hazards Map, Leighton (1990), indicates the site does not lie within mapped inundation boundaries due to a seiche or a breached upgradient reservoir.

Landsliding

The probability of seismically-induced landslides occurring on the site is considered to be low due to the relatively flat topographic gradient across or adjacent to the site.

Based on the Site Plan prepared by Nadel Architects, it is anticipated that the proposed mixed-use structures will be located a minimum of 20 feet away from the existing flood control channel. Since the channel is approximately 8 to 9 feet in depth, it is the opinion of this firm that the proposed mixed-use structures would not be affected in the event that the channel's retaining walls failed.

CONCLUSIONS AND RECOMMENDATIONS

Based upon the exploration, laboratory testing, and research, it is the finding of Geotechnologies, Inc. that construction of the proposed mixed-use structures, vehicular bridge, and pedestrian bridge, are considered feasible from a geotechnical engineering standpoint provided the advice and recommendations presented herein are followed and implemented during construction.

Between 1 and 12½ feet of fill materials were encountered during exploration. The existing fill is not suitable for support of the proposed foundations, concrete slabs or paving, but may be reused for the construction of uniform compacted fill pads.

This firm recommends that the proposed at-grade mixed-use structures should be supported on conventional foundations bearing in a uniform compacted fill pad. For the construction of a uniform compacted fill pad, all existing fill materials and upper native soils found within the footprint of the proposed structures should be removed and recompacted to a minimum depth of 5 feet below the existing grade, or 3 feet below the bottom of the proposed foundations, whichever is greater. In addition, the compacted fill pad should extend horizontally a minimum of 3 feet beyond the edge of the proposed foundations, or for a distance equal to the depth of the compacted fill below the foundations, whichever is greater.

This firm recommends that the proposed below-grade buildings may be supported in native soils found below 5 feet in depth. Excavation of the proposed subterranean level will remove the unsuitable materials in these building areas.

It is anticipated that the foundations to support the proposed vehicular bridge and pedestrian bridge will be constructed adjacent to the existing flood control channel retaining walls. In order to avoid surcharging the channel retaining walls, this firm recommends that both bridge structures be supported on a deepened foundation system, consisting of drilled cast-in-placed concrete friction piles. The piles should derive their support from undisturbed alluvial soils found below a 1:1 (h:v) surcharge plane projected upward from the bottom of the adjacent channel retaining wall.

The validity of the conclusions and design recommendations presented herein is dependent upon review of the geotechnical aspects of the proposed construction by this firm. The subsurface conditions described herein have been projected from excavations on the site as indicated and should in no way be construed to reflect any variations which may occur between these excavations or which may result from changes in subsurface conditions. Any changes in the design or location of any structure, as outlined in this report, should be reviewed by this office. The recommendations contained herein should not be considered valid until reviewed and modified or reaffirmed subsequent to such review.

FILL SOILS

The site has a relatively uniform fill depth of 2½ feet. However, deeper fill as much as 12½ feet was encountered in a few of the borings. This material and any fill generated during demolition should be removed and recompacted as controlled fill prior to foundation excavation.

EXPANSIVE SOILS

The onsite geologic materials are in the very low to moderate expansion range. The Expansion Index ranges between 13 and 66 for representative bulk samples. Recommended reinforcing is noted in the "Foundation Design" and "Slabs on Grade" sections of this report.

SOIL CORROSION POTENTIAL

The results of the soil corrosivity testing performed on a bulk sample representative of the onsite soils by HDR Schiff Associates indicate that the electrical resistivities of the soils are in the mildly corrosive category in their field moisture condition, and in the moderately corrosive category when saturated. The soil pH value of the sample was 7.5, which is considered to be mildly alkaline. The soluble salt content was low.

In summary, the site soils are classified as moderately corrosive to ferrous materials. Special cement types need not be utilized for concrete structures in contact with the soils, since the sulfate content of the soils is negligible. Detailed results, discussion of results and recommended mitigating measures are provided within the enclosed HDR Schiff report dated August 13, 2013.

Based on the moderately corrosive characteristics of the site soils, it is anticipated that buried ferrous metal pipe would corrode and deteriorate prematurely if used at the project site. Therefore, it is the recommendation of this firm that ABS or PVC pipe should be utilized rather than ferrous metal pipe for utilities underlying the subject site.

GRADING GUIDELINES

Site Preparation

- A thorough search should be made for possible underground utilities and/or structures. Any existing or abandoned utilities or structures located within the footprint of the proposed grading should be removed or relocated as appropriate.

- All vegetation, existing fill, and soft or disturbed geologic materials should be removed from the areas to receive controlled fill. All existing fill materials and any disturbed geologic materials resulting from grading operations shall be completely removed and properly recompacted prior to foundation excavation.

- Any vegetation or associated root system located within the footprint of the proposed structures should be removed during grading.

- Subsequent to the indicated removals, the exposed grade shall be scarified to a depth of six inches, moistened to optimum moisture content, and recompacted in excess of the minimum required comparative density.

- The excavated areas shall be observed by the geotechnical engineer prior to placing compacted fill.

Recommended Overexcavation and Blending

The proposed building areas shall be excavated to a minimum depth of 5 feet below the bottom of the existing grade, or 3 feet below the bottom of the proposed foundations, whichever is greater. The excavation shall extend at least three feet beyond the edge of foundations, or for a distance equal to the depth of fill below the foundations, whichever is greater. It is very important that the positions of the proposed structures are accurately located so that the limits of the graded area are accurate and the grading operation proceeds efficiently.

Once the onsite soils have been removed it is recommended that they should be well blended to reduce the overall expansion index of the newly placed controlled fill. Where the site grading will result in a net export, the sandier or more granular materials should be segregated from the stockpiled soils and the more clayey or expansive materials should be exported. Samples of the segregated and/or blended soils should be tested by this office to ascertain the expansion index prior to placement and compaction.

Compaction

The City of Los Angeles Department of Building and Safety requires a minimum comparative compaction of 95 percent of the laboratory maximum density where the soils to be utilized in the fill have less than 15 percent finer than 0.005 millimeters.

All fill should be mechanically compacted in layers not more than 8 inches thick. All fill shall be compacted to at least 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum laboratory density for the materials used. The maximum density shall be determined by the laboratory operated by Geotechnologies, Inc. using the test method described in the most recent revision of ASTM D 1557.

Field observation and testing shall be performed by a representative of the geotechnical engineer during grading to assist the contractor in obtaining the required degree of compaction and the proper moisture content. Where compaction is less than required, additional compactive effort shall be made with adjustment of the moisture content, as necessary, until a minimum of 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) compaction is obtained.

Acceptable Materials

The excavated onsite materials, both existing fill an alluvium, are considered satisfactory for reuse in the controlled fills as long as any debris, organic matter, or oversized materials are removed.

Any imported materials shall be observed and tested by the representative of the geotechnical engineer prior to use in fill areas. Imported materials should contain sufficient fines so as to be relatively impermeable and result in a stable subgrade when compacted. Any required import materials should consist of geologic materials with an expansion index of less than 50. The water-soluble sulfate content of the import materials should be less than 0.1% percentage by weight.

Imported materials should be free from chemical or organic substances which could affect the proposed development. A competent professional should be retained in order to test imported materials and address environmental issues and organic substances which might affect the proposed development.

Utility Trench Backfill

Utility trenches should be backfilled with controlled fill. The utility should be bedded with clean sands at least one foot over the crown. The remainder of the backfill may be onsite soil compacted to 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the laboratory maximum density. Utility trench backfill should be tested by representatives of this firm in accordance with the most recent revision of ASTM D-1557.

Shrinkage

Shrinkage results when a volume of soil removed at one density is compacted to a higher density. A shrinkage factor between 10 and 20 percent should be anticipated when excavating and recompacting the existing fill and underlying native geologic materials on the site to an average comparative compaction of 92 percent.

Weather Related Grading Considerations

When rain is forecast all fill that has been spread and awaits compaction shall be properly compacted prior to stopping work for the day or prior to stopping due to inclement weather. These fills, once compacted, shall have the surface sloped to drain to an area where water can be removed.

Temporary drainage devices should be installed to collect and transfer excess water to the street in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. Drainage should not be allowed to flow uncontrolled over any descending slope.

Work may start again, after a period of rainfall, once the site has been reviewed by a representative of this office. Any soils saturated by the rain shall be removed and aerated so that the moisture content will fall within three percent of the optimum moisture content.

Surface materials previously compacted before the rain shall be scarified, brought to the proper moisture content and recompacted prior to placing additional fill, if considered necessary by a representative of this firm.

Abandoned Seepage Pits

No abandoned seepage pits were encountered during exploration and none are known to exist on the site. However, should such a structure be encountered during grading, options to permanently abandon seepage pits include complete removal and backfill of the excavation with compacted fill, or drilling out the loose materials and backfilling to within a few feet of grade with slurry, followed by a compacted fill cap.

If the subsurface structures are to be removed by grading, the entire structure should be demolished. The resulting void may be refilled with compacted soil. Concrete and brick generated during the seepage pit removal may be reused in the fill as long as all fragments are less than 6 inches in longest dimension and the debris comprises less than 15 percent of the fill by volume. All grading should comply with the recommendations of this report.

Where the seepage pit structure is to be left in place, the seepage pits should cleaned of all soil and debris. This may be accomplished by drilling. The pits should be filled with minimum 1-1/2 sack concrete slurry to within 5 feet of the bottom of the proposed foundations. In order to provide a more uniform foundation condition, the remainder of the void should be filled with controlled fill.

Geotechnical Observations and Testing During Grading

Geotechnical observations and testing during grading are considered to be a continuation of the geotechnical investigation. It is critical that the geotechnical aspects of the project be reviewed by representatives of Geotechnologies, Inc. during the construction process. Compliance with the design concepts, specifications or recommendations during construction requires review by this firm during the course of construction. Any fill which is placed should be observed, tested, and verified if used for engineered purposes. Please advise this office at least twenty-four hours prior to any required site visit.

LEED Considerations

The Leadership in Energy and Environmental Design (LEED) Green Building Rating System encourages adoption of sustainable green building and development practices. Credit for LEED Certification can be assigned for reuse of construction waste and diversion of materials from landfills in new construction.

In an effort to provide the design team with a viable option in this regard, demolition debris could be crushed onsite in order to use it in the ongoing grading operations. The environmental ramifications of this option, if any, should be considered by the team.

The demolition debris should be limited to concrete, asphalt and other non-deleterious materials. All deleterious materials should be removed including, but not limited to, paper, garbage, ceramic materials and wood.

For structural fill applications, the materials should be crushed to 2 inches in maximum dimension or smaller. The crushed materials should be thoroughly blended and mixed with onsite soils prior to placement as compacted fill. The amount of crushed material should not exceed 20 percent. The blended and mixed materials should be tested by this office prior to placement to insure it is suitable for compaction purposes. The blended and mixed materials should be tested by Geotechnologies, Inc. during placement to insure that it has been compacted in a suitable manner.

FOUNDATION DESIGN

CONVENTIONAL FOUNDATIONS FOR PROPOSED MIXED-USE STRUCTURES

The proposed at-grade mixed-use structures may be supported by conventional foundations bearing in a newly compacted fill pad. The proposed structures which will be serviced by below-grade parking may be supported in native soils found below 5 feet in depth. All conventional foundations for a structure should bear in the same material.

Continuous foundations may be designed for a bearing capacity of 2,500 pounds per square foot, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended compacted fill pad.

Column foundations may be designed for a bearing capacity of 3,000 pounds per square foot, and should be a minimum of 24 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended compacted fill pad.

The bearing capacity increase for each additional foot of width is 75 pounds per square foot. The bearing capacity increase for each additional foot of depth is 300 pounds per square foot. The maximum recommended bearing capacity is 5,000 pounds per square foot.

The bearing capacities indicated above are for the total of dead and frequently applied live loads, and may be increased by one third for short duration loading, which includes the effects of wind or seismic forces.

Miscellaneous Foundations

Conventional foundations for structures such as privacy walls or trash enclosures which will not be rigidly connected to the proposed structures may be deepened through any existing fill in order to bear in shallow undisturbed alluvial soils. Continuous footings may be designed for a bearing capacity of 1,500 pounds per square foot, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended alluvial soils. No bearing capacity increases are recommended.

Since the recommended bearing capacity is a net value, the weight of concrete in the foundations may be taken as 50 pounds per cubic foot and the weight of the soil backfill may be neglected when determining the downward load on the foundations.

Foundation Reinforcement

All continuous foundations should be reinforced with a minimum of four #4 steel bars. Two should be placed near the top of the foundation, and two should be placed near the bottom.

Lateral Design

Resistance to lateral loading may be provided by friction acting at the base of foundations and by passive earth pressure. An allowable coefficient of friction of 0.3 may be used with the dead load forces.

Passive geologic pressure for the sides of foundations poured against undisturbed or recompacted soil may be computed as an equivalent fluid having a density of 300 pounds per cubic foot with a maximum earth pressure of 3,000 pounds per square foot.

The passive and friction components may be combined for lateral resistance without reduction. A one-third increase in the passive value may be used for short duration loading such as wind or seismic forces.

Foundation Settlement

Settlement of the foundation system is expected to occur on initial application of loading. The maximum settlement is expected to be 1 inch and occur below the heaviest loaded columns. Differential settlement is not expected to exceed ½ inch. As a general guideline, whatever the column spacing may be for the building is the distance over which the settlement may occur.

FRICTION PILES FOR PROPOSED VEHICULAR AND PEDESTRIAN BRIDGES

It is anticipated that the foundations to support the proposed vehicular bridge and pedestrian bridge will be constructed adjacent to the existing flood control channel. In order to avoid surcharging the channel retaining walls, this firm recommends that both bridge structures be supported on a deepened foundation system, consisting of drilled cast-in-placed concrete friction piles. The piles shall penetrate through any existing fill materials in order to derive their strength exclusively from undisturbed alluvial soils.

The piles should derive their support below a 1:1 (h:v) surcharge plane projected upward from the bottom of the adjacent channel retaining wall. The portion of the piles located above the 1:1 (h:v) surcharge plane must be sleeved in order to prevent surcharging the adjacent retaining wall. This condition is illustrated in the enclosed L-PILE printouts. For the sleeved portion of the pile, a round concrete form should be used. The form is placed prior to concrete pouring thereby impeding the bonding between concrete and soils from the non-bearing strata.

Vertical Capacities

The vertical capacities of 18 and 24 drilled cast-in-place piles are shown in the enclosed “Drilled Cast in Place Pile Capacities” charts. For illustrational purposes, capacities were calculated for scenarios where the upper 6 feet of the proposed piles would be above the 1:1 surcharge plane, and therefore would be sleeved. Capacities based on dead plus live load are indicated. A one-third increase may be used for transient loading such as wind or seismic forces. The capacities presented are based on the strength of the soils. The compressive and tensile strength of the pile sections should be checked to verify the structural capacity of the piles.

Piles in groups should be spaced at least 3 diameters on center. If the piles are so spaced, no reduction in the downward or upward capacities need be considered due to group action.

Lateral Design

Lateral loads may be resisted by the piles embedded into the underlying alluvial soils. In order to avoid surcharging the existing flood control channel walls, the proposed piles shall be designed to derive lateral resistance from the alluvial soils below a 1:1 (h:v) surcharge plane projected upward from the bottom of the channel walls. It is recommended that the maximum pile deflection be limited to ½-inch.

Resistance to lateral loading may be provided by passive earth pressure. Passive earth pressure for the sides of foundations poured against undisturbed native soils may be computed as an equivalent fluid having a density of 300 pounds per cubic foot.

Analyses of the proposed piles using varying shear loads were performed using the program LPILE Plus (version 4.0) included in the Appendix of this report. The printouts show the calculated shear, moment, and deflection for single, isolated caissons, with diameters of 18 and

24 inches. A scenario where the upper 6 feet of the pile is found above the 1:1 surcharge plane, and is therefore sleeved, was analyzed. No factors of safety have been applied to the lateral load values calculated to induce the lateral deflection. Assumed as part of these lateral capacity calculations are:

- A Free Head Condition
- A 75 kip vertical loads
- A concrete modulus of elasticity of 3,605,000 pounds per square inch (psi)
- A lateral shear loading range of 2, 5 and 10 kips

If any of these assumptions are not valid, please contact this firm and a modified analysis can be performed.

Pile Installation

Caving may occur during drilling of the proposed piles due to the granular nature of some of the soil layers underlying the site. If caving occurs during drilling, casing will be required in order to achieve the required depth and maintain an open hole to allow the placement of the steel and concrete. If casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than 5 feet. Pile excavations should be filled with concrete as soon after drilling and inspection as possible; the shafts should not be left open overnight.

Settlement

The maximum settlement of the proposed piles is not expected to exceed ¼-inch.

Foundation Observations

It is critical that all foundation excavations are observed by a representative of this firm to verify penetration into the recommended bearing materials. The observation should be performed prior to the placement of reinforcement. Foundations should be deepened to extend into satisfactory geologic materials, if necessary.

Foundation excavations should be cleaned of all loose soils prior to placing steel and concrete. Any required foundation backfill should be mechanically compacted, flooding is not permitted.

POOL SHELL DESIGN

Based on a site plan provided to this firm, three pool structures are anticipated as part of the proposed development. The anticipated depth of the pools is not known at this time. The proposed pools may be supported on undisturbed alluvial soils, or properly compacted fill materials. This firm recommends that each individual pool structure bears in the same material.

A bearing capacity of 3,000 pounds per square foot may be assigned to the compacted fill or undisturbed alluvial soils to support the proposed pool. The pool shell walls should be designed free-standing. Exterior pool walls, up to 6 feet in height, should be designed to resist a triangular equivalent fluid pressure of 30 pcf.

RETAINING WALL DESIGN

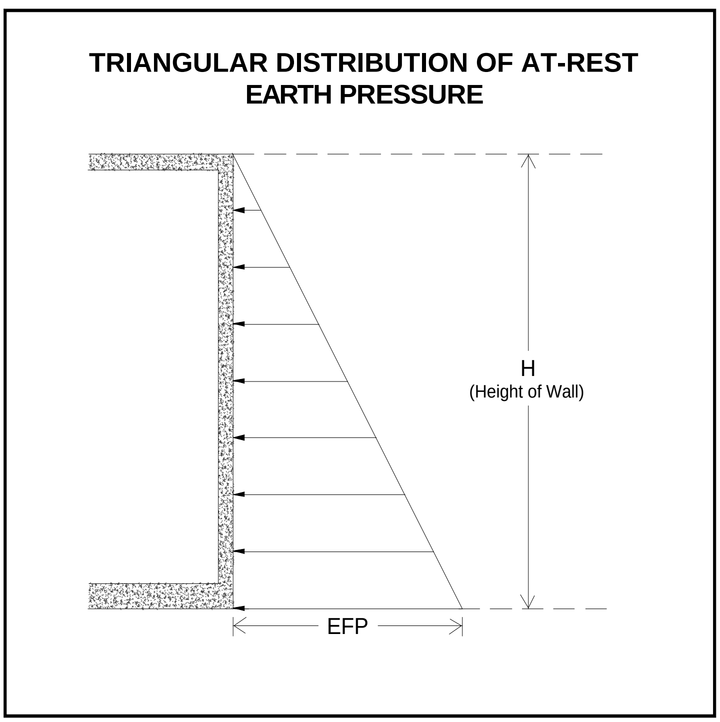
Cantilever Retaining Walls

Miscellaneous cantilever retaining walls supporting a level backslope may be designed utilizing a triangular distribution of pressure. Cantilever retaining walls may be designed for 30 pounds per cubic foot for walls retaining up to 6 feet in height. Cantilever retaining walls may be designed for 45 pounds per cubic foot for walls retaining up to 15 feet in height.

For this equivalent fluid pressure to be valid, walls which are to be restrained at the top should be backfilled prior to the upper connection being made. Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures.

Restrained Drained Retaining Walls

Restrained retaining walls may be designed to resist a triangular pressure distribution of at-rest earth pressure as indicated in the diagram below. The at-rest pressure for design purposes would be 66.3 pounds per cubic foot. Additional earth pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures.



In addition to the recommended earth pressure, the upper ten feet of the retaining wall adjacent to streets, driveways or parking areas should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot surcharge behind the walls due to normal street traffic. If the traffic is kept back at least ten feet from the retaining walls, the traffic surcharge may be neglected.

The lateral earth pressures recommended above for retaining walls assume that a permanent drainage system will be installed so that external water pressure will not be developed against the walls. Also, where necessary, the retaining walls should be designed to accommodate any surcharge pressures that may be imposed by existing buildings on the adjacent property.

Dynamic (Seismic) Earth Pressure

The maximum dynamic active pressure is equal to the sum of the initial static pressure and the dynamic (seismic) pressure increment. Under the most recent building code, as interpreted by most building departments, seismic earth pressure is required in the design of restraining walls which support over 12 feet of earth.

The combined lateral active and seismic earth pressures imposed on basement walls retaining up to 15 feet of earth may be taken as an equivalent fluid with a density of 65.3 pounds per cubic foot (i.e. 45 pcf active lateral soil pressure + 20.3 pcf seismic increment). Based on this consideration, it is recommended the proposed basement walls be designed to resist the more conservative "at-rest" earth pressure of 66.3 pounds per cubic foot.

Surcharge from Adjacent Structures

Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures for retaining walls and shoring design.

The following surcharge equation provided in the LADBS Information Bulletin Document No. P/BC 2008-83, may be utilized to determine the surcharge loads on basement walls and shoring system for existing structures located within the 1:1 (h:v) surcharge influence zone of the excavation and basement.

Resultant lateral force: $R = (0.3*P*h^2)/(x^2+h^2)$

Location of lateral resultant: $d = x*[(x^2/h^2+1)*\tan^{-1}(h/x)-(x/h)]$

where:

R	=	resultant lateral force measured in pounds per foot of wall width.
P	=	resultant surcharge loads of continuous or isolated footings measured in pounds per foot of length parallel to the wall.
x	=	distance of resultant load from back face of wall measured in feet.
h	=	depth below point of application of surcharge loading to top of wall footing measured in feet.
d	=	depth of lateral resultant below point of application of surcharge loading measure in feet.
$\tan^{-1}(h/x)$	=	the angle in radians whose tangent is equal to h/x.

The structural engineer and shoring engineer may use this equation to determine the surcharge loads based on the loading of the adjacent structures located within the surcharge influence zone.

Retaining Wall Drainage

All retaining walls shall be provided with a subdrain system in order to minimize the potential for future hydrostatic pressure buildup behind the proposed retaining walls. Subdrains may consist of four-inch diameter perforated pipes, placed with perforations facing down. The pipe shall be encased in at least one-foot of gravel around the pipe. The gravel shall be wrapped in filter fabric. The gravel may consist of three-quarter inch to one inch crushed rocks.

As an alternative to the standard perforated subdrain pipe and gravel drainage system, the use of gravel pockets and weepholes is an acceptable drainage method. Weepholes shall be a minimum of 2 inches in diameter, placed at 8 feet on center along the base of the wall. Gravel pockets shall be a minimum of 1 cubic foot in dimension, and may consist of three-quarter inch to one inch crushed rocks, wrapped in filter fabric. A collector pipe shall be installed to direct collected waters to a sump.

Certain types of subdrain pipe are not acceptable to the various municipal agencies, it is recommended that prior to purchasing subdrainage pipe, the type and brand is cleared with the proper municipal agencies. Subdrainage pipes should outlet to an acceptable location. Some municipalities do not allow the use of flat-drainage products, such as Miradrain. The use of such a product should be researched with the building official. The City of Los Angeles only allows the use of flat drainage products when in conjunction with a conventional perforated subdrain pipe and gravel, or gravel pockets and weepholes.

The lateral earth pressures recommended above for retaining walls assume that a permanent drainage system will be installed so that external water pressure will not be developed against the walls. If a drainage system is not provided, the walls should be designed to resist an external hydrostatic pressure due to water in addition to the lateral earth pressure. In any event, it is recommended that retaining walls be waterproofed.

Sump Pump Design

The purpose of the recommended retaining wall backdrainage system is to relieve hydrostatic pressure. Groundwater was not encountered during exploration to a depth of 50 feet which corresponds to 35 feet below the base of the proposed structure. Therefore the only water which could effect the proposed retaining walls would be irrigation waters and precipitation. Additionally, the proposed site grading is such that all drainage is directed to the street and the structure has been designed with adequate non-erosive drainage devices.

Based on these considerations the retaining wall backdrainage system is not expected to experience an appreciable flow of water, and in particular, no groundwater will effect it. However, for the purposes of design, a flow of 5 gallons per minute may be assumed.

Waterproofing

Moisture effecting retaining walls is one of the most common post construction complaints. Poorly applied or omitted waterproofing can lead to efflorescence or standing water inside the building. Efflorescence is a process in which a powdery substance is produced on the surface of the concrete by the evaporation of water. The white powder usually consists of soluble salts such as gypsum, calcite, or common salt. Efflorescence is common to retaining walls and does not affect their strength or integrity.

It is recommended that retaining walls be waterproofed. Waterproofing design and inspection of its installation is not the responsibility of the geotechnical engineer. A qualified waterproofing consultant should be retained in order to recommend a product or method which would provide protection to below grade walls.

Retaining Wall Backfill

Any required backfill should be mechanically compacted in layers not more than 8 inches thick, to at least 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) relative compaction, obtainable by the most recent revision of ASTM D 1557 method of compaction. Flooding should not be permitted. Compaction within 5 feet, measured horizontally, behind a retaining structure should be achieved by use of light weight, hand operated compaction equipment.

Proper compaction of the backfill will be necessary to reduce settlement of overlying walks and paving. Some settlement of required backfill should be anticipated, and any utilities supported therein should be designed to accept differential settlement.

TEMPORARY EXCAVATIONS

Based on the depth of fill encountered during exploration, it is anticipated that excavations to a depth of 12½ will be required for the recommended removal and recompaction and up to 15 feet for the proposed subterranean parking levels. The excavations are expected to expose fill and dense native soils, which are suitable for vertical excavations up to 5 feet where not surcharged by adjacent traffic or structures. Excavations which will be surcharged must be shored.

Where sufficient space is available, temporary unsurcharged embankments could be cut at a uniform 1:1 slope gradient to a maximum height of 15 feet. A uniform sloped excavation is sloped from bottom to top and does not have a vertical component.

Where sloped embankments are utilized, the tops of the slopes should be barricaded to prevent vehicles and storage loads near the top of slope within a horizontal distance equal to the depth of the excavation. If the temporary construction embankments are to be maintained during the rainy season, berms are strongly recommended along the tops of the slopes to prevent runoff water from entering the excavation and eroding the slope faces. Water should not be allowed to pond on top of the excavation nor to flow towards it.

Excavation Observations

It is critical that the soils exposed in the cut slopes are observed by a representative of Geotechnologies, Inc. during excavation so that modifications of the slopes can be made if variations in the geologic material conditions occur. Many building officials require that temporary excavations should be made during the continuous observations of the geotechnical engineer. All excavations should be stabilized within 30 days of initial excavation.

SHORING DESIGN

The following information on the design and installation of the shoring is as complete as possible at this time. It is suggested that Geotechnologies, Inc. review the final shoring plans and specifications prior to bidding or negotiating with a shoring contractor.

One method of shoring would consist of steel soldier piles, placed in drilled holes and backfilled with concrete. Another method of shoring consists of steel soldier piles vibrated into place. Either of these methods is acceptable to Geotechnologies, Inc. The soldier piles may be designed as cantilevers or laterally braced utilizing drilled tied-back anchors or raker braces.

Soldier Piles

Drilled cast-in-place soldier piles should be placed no closer than 2 diameters on center. The minimum diameter of the piles is 18 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wideflange section. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the wideflange section to the geologic materials. For design purposes, an allowable passive value for the geologic materials below the bottom plane of excavation, may be assumed to be 600 pounds per square foot per foot. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed geologic materials.

Casing may be required should caving be experienced in the granular geologic materials. If casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than 5 feet.

The frictional resistance between the soldier piles and retained geologic material may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.3 based on uniform contact between the steel beam and lean-mix concrete and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 500 pounds per square foot. The minimum depth of embedment for shoring piles is 5 feet below the bottom of the footing excavation or 7 feet below the bottom of excavated plane whichever is deeper.

Lagging

Soldier piles and anchors should be designed for the full anticipated pressures. Due to arching in the geologic materials, the pressure on the lagging will be less. It is recommended that the lagging should be designed for the full design pressure but be limited to a maximum of 400 pounds per square foot. It is recommended that a representative of this firm observe the installation of lagging to insure uniform support of the excavated embankment.

Tied-Back Anchors

Tied-back anchors may be used to resist lateral loads. Friction anchors are recommended. For design purposes, it may be assumed that the active wedge adjacent to the shoring is defined by a plane drawn 35 degrees with the vertical through the bottom plane of the excavation. Friction anchors should extend a minimum of 20 feet beyond the potentially active wedge.

Drilled friction anchors may be designed for a skin friction of 600 pounds per square foot. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. This skin friction is based on 15 foot high shoring, a tied back anchor elevation 6 feet below grade and a minimum twenty foot embedment beyond the potentially active wedge yielding an overburden of 12½ feet below ground surface. Where belled anchors are utilized, the capacity of belled anchors may be designed by applying the skin friction over the surface area of the bonded anchor shaft. The diameter of the bell may be utilized as the diameter of the bonded anchor shaft when determining the surface area. This implies that in order for the belled anchor to fail, the entire parallel soil column must also fail.

Depending on the techniques utilized, and the experience of the contractor performing the installation, it is anticipated that a skin friction of 2,500 pounds per square foot could be utilized for post-grouted anchors. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads.

Anchors should be placed at least 6 feet on center to be considered isolated. It is recommended that at least 3 of the initial anchors have their capacities tested to 200 percent of their design capacities for a 24-hour period to verify their design capacity.

The total deflection during this test should not exceed 12 inches. The anchor deflection should not exceed 0.75 inches during the 24 hour period, measured after the 200 percent load has been applied. All anchors should be tested to at least 150 percent of design load. The total deflection during this test should not exceed 12 inches.

The rate of creep under the 150 percent test load should not exceed 0.1 inch over a 15 minute period in order for the anchor to be approved for the design loading. After a satisfactory test, each anchor should be locked-off at the design load. This should be verified by rechecking the load in the anchor. The load should be within 10 percent of the design load. Where satisfactory tests are not attained, the anchor diameter and/or length should be increased or additional anchors installed until satisfactory test results are obtained. The installation and testing of the anchors should be observed by the geotechnical engineer. Minor caving during drilling of the anchors should be anticipated.

Anchor Installation

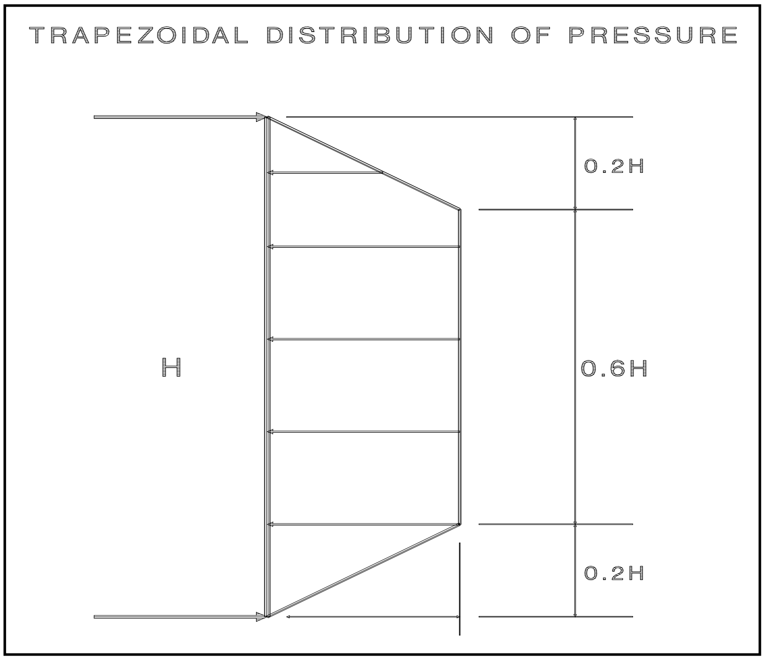
Tied-back anchors may be installed between 20 and 40 degrees below the horizontal. Caving of the anchor shafts, particularly within sand deposits, should be anticipated and the following provisions should be implemented in order to minimize such caving. The anchor shafts should be filled with concrete by pumping from the tip out, and the concrete should extend from the tip of the anchor to the active wedge. In order to minimize the chances of caving, it is recommended that the portion of the anchor shaft within the active wedge be backfilled with sand before testing the anchor. This portion of the shaft should be filled tightly and flush with the face of the excavation. The sand backfill should be placed by pumping; the sand may contain a small amount of cement to facilitate pumping.

Lateral Pressures

Cantilevered shoring supporting a level backslope may be designed utilizing a triangular distribution of pressure as indicated in the following table:

HEIGHT OF SHORING “H” (feet)	EQUIVALENT FLUID PRESSURE (pounds per cubic foot)
Up to 15	36

A trapezoidal distribution of lateral earth pressure would be appropriate where shoring is to be restrained at the top by bracing or tie backs, with the trapezoidal distribution as shown in the diagram below.



Restrained shoring supporting a level backslope may be designed utilizing a trapezoidal distribution of pressure as indicated in the following table:

HEIGHT OF SHORING "H" (feet)	DESIGN SHORING FOR (Where H is the height of the wall)
Up to 15	23H

Where a combination of sloped embankment and shoring is utilized, the pressure will be greater and must be determined for each combination. Additional active pressure should be applied where the shoring will be surcharged by adjacent traffic or structures. Where a combination of sloped embankment and shoring is utilized, the pressure will be greater and must be determined for each combination.

Deflection

It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized that some deflection will occur. It is estimated that the deflection could be on the order of one inch at the top of the shored embankment. If greater deflection occurs during construction, additional bracing may be necessary to minimize settlement of adjacent buildings and utilities in adjacent street and alleys. If desired to reduce the deflection, a greater active pressure could be used in the shoring design. Where internal bracing is used, the rakers should be tightly wedged to minimize deflection. The proper installation of the raker braces and the wedging will be critical to the performance of the shoring.

The City of Los Angeles Department of Building and Safety requires limiting shoring deflection to ½ inch at the top of the shored embankment where a structure is within a 1:1 plane projected up from the base of the excavation. A maximum deflection of 1-inch has been allowed provided there are no structures within a 1:1 plane drawn upward from the base of the excavation.

Monitoring

Because of the depth of the excavation, some mean of monitoring the performance of the shoring system is suggested. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all soldier piles and the lateral movement along the entire lengths of selected soldier piles. Also, some means of periodically checking the load on selected anchors will be necessary, where applicable.

Some movement of the shored embankments should be anticipated as a result of the relatively deep excavation. It is recommended that photographs of the existing buildings on the adjacent properties be made during construction to record any movements for use in the event of a dispute.

Shoring Observations

It is critical that the installation of shoring is observed by a representative of Geotechnologies, Inc. Many building officials require that shoring installation should be performed during continuous observation of a representative of the geotechnical engineer. The observations insure that the recommendations of the geotechnical report are implemented and so that modifications of the recommendations can be made if variations in the geologic material or groundwater conditions warrant. The observations will allow for a report to be prepared on the installation of shoring for the use of the local building official, where necessary.

Raker Brace Foundations

An allowable bearing pressure of 4,000 pounds per square foot may be used for the design a raker foundations. This bearing pressure is based on a raker foundation a minimum of 4 feet in width and length as well as 4 feet in depth. The base of the raker foundations should be horizontal. Care should be employed in the positioning of raker foundations so that they do not interfere with the foundations for the proposed structure.

SLABS ON GRADE

Concrete Slabs-on Grade

Concrete floor slabs should be a minimum of 5 inches in thickness. Slabs-on-grade should be cast over properly controlled fill materials. Any geologic materials loosened or over-excavated should be wasted from the site or properly compacted to 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum dry density.

Outdoor concrete flatwork should be a minimum of 4 inches in thickness. Outdoor concrete flatwork should be cast over undisturbed alluvial soils, or properly controlled fill materials. Any geologic materials loosened or over-excavated should be wasted from the site or properly compacted to 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum dry density.

Design of Slabs That Receive Moisture-Sensitive Floor Coverings

Geotechnologies, Inc. does not practice in the field of moisture vapor transmission evaluation and mitigation. Therefore it is recommended that a qualified consultant be engaged to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. The qualified consultant should provide recommendations for mitigation of potential adverse impacts of moisture vapor transmission on various components of the structure.

Where dampness would be objectionable, it is recommended that the floor slabs should be waterproofed. A qualified waterproofing consultant should be retained in order to recommend a product or method which would provide protection for concrete slabs-on-grade.

All concrete slabs-on-grade should be supported on vapor retarder. The design of the slab and the installation of the vapor retarder should comply with the most recent revisions of ASTM E 1643 and ASTM E 1745. Where a vapor retarder is used, a low-slump concrete should be used to minimize possible curling of the slabs. The barrier can be covered with a layer of trimable, compactible, granular fill, a minimum of 2 inches in thickness, where it is thought to be beneficial. See ACI 302.2R-32, Chapter 7 for information on the placement of vapor retarders and the use of a fill layer.

Concrete Crack Control

The recommendations presented in this report are intended to reduce the potential for cracking of concrete slabs-on-grade due to settlement. However even where these recommendations have been implemented, foundations, stucco walls and concrete slabs-on-grade may display some cracking due to minor soil movement and/or concrete shrinkage. The occurrence of concrete cracking may be reduced and/or controlled by limiting the slump of the concrete used, proper concrete placement and curing, and by placement of crack control joints at reasonable intervals, in particular, where re-entrant slab corners occur.

For standard control of concrete cracking, a maximum crack control joint spacing of 12 feet should not be exceeded. Lesser spacings would provide greater crack control. Joints at curves and angle points are recommended. The crack control joints should be installed as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. Construction joints should be designed by a structural engineer.

Complete removal of the existing fill soils beneath outdoor flatwork such as walkways or patio areas, is not required, however, due to the rigid nature of concrete, some cracking, a shorter design life and increased maintenance costs should be anticipated. In order to provide uniform support beneath the flatwork it is recommended that a minimum of 12 inches of the exposed subgrade beneath the flatwork be scarified and recompacted to 90 percent relative compaction.

Slab Reinforcing

Concrete slabs-on-grade should be reinforced with a minimum of #4 steel bars on 16-inch centers each way. Outdoor flatwork should be reinforced with a minimum of #3 steel bars on 18-inch centers each way.

PAVEMENTS

Prior to placing paving, the existing grade should be scarified to a depth of 12 inches, moistened as required to obtain optimum moisture content, and recompacted to 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum density as determined by the most recent revision of ASTM D 1557. The client should be aware that removal of all existing fill in the area of new paving is not required, however, pavement constructed in this manner will most likely have a shorter design life and increased maintenance costs. The following pavement sections are recommended:

Service	Asphalt Pavement Thickness Inches	Base Course Inches
Passenger Cars (TI=4)	3	4
Moderate Truck (TI=6)	4	6

Aggregate base should be compacted to a minimum of 95 percent of the most recent revision of ASTM D 1557 laboratory maximum dry density. Base materials should conform to Sections 200-2.2 or 200-2.4 of the “Standard Specifications for Public Works Construction”, (Green Book), latest edition.

Concrete paving may also be utilized for the project. For concrete paving sections to be subject to passenger cars and medium truck traffic, concrete paving shall be a minimum of 6 inches in thickness, and shall be underlain by 4 inches of aggregate base. For standard crack control maximum expansion joint spacing of 12 feet should not be exceeded. Lesser spacings would provide greater crack control. Joints at curves and angle points are recommended.

The performance of pavement is highly dependent upon providing positive surface drainage away from the edges. Ponding of water on or adjacent to pavement can result in saturation of the subgrade materials and subsequent pavement distress. If planter islands are planned, the perimeter curb should extend a minimum of 12 inches below the bottom of the aggregate base.

SITE DRAINAGE

Proper surface drainage is critical to the future performance of the project. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the designed engineering properties. Proper site drainage should be maintained at all times.

All site drainage, with the exception of any required to disposed of onsite by stormwater regulations, should be collected and transferred to the street in non-erosive drainage devices. The proposed structure should be provided with roof drainage. Discharge from downspouts, roof drains and scuppers should not be permitted on unprotected soils within five feet of the building perimeter. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. Drainage should not be allowed to flow uncontrolled over any descending slope. Planters which are located within a distance equal to the depth of a retaining wall should be sealed to prevent moisture adversely affecting the wall. Planters which are located within five feet of a foundation should be sealed to prevent moisture effecting the earth materials supporting the foundation.

STORMWATER DISPOSAL

Recently regulatory agencies have been requiring the disposal of a certain amount of stormwater generated on a site by infiltration into the site soils. Increasing the moisture content of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the designed engineering properties. This means that any overlying structure, including buildings, pavements and concrete flatwork, could sustain damage due to saturation of the subgrade soils. Structures serviced by subterranean levels could be adversely impacted by stormwater disposal by increasing the design fluid pressures on retaining walls and causing leaks in the walls. Proper site drainage is critical to the performance of any structure in the built environment.

This firm conducted percolation testing on the site for the preparation of a previous geotechnical investigation dated May 28, 2008. The location of the percolation test pit (Test Pit 1) is shown in the attached Plot Plan. The test pit was excavated to a depth of 6 feet with the aid of hand tools. A log showing the geologic materials encountered in the test pit may be found in the Appendix of this report. The test pit was presoaked for a minimum of 2 hours prior to the test. After the presoak, the test pit was refilled with water and the absorption of the soils was measured.

Based on results of the percolation test, a percolation rate of 6 inches per hour may be utilized for design purposes. This rate is based on the alluvial soils encountered in the test pit at a depth of 6 feet. It is recommended that stormwater should only percolate into native alluvial soils. It should be noted that the recommended percolation rate is based on testing at discrete locations and the overall percolation rate of the system could vary considerably.

This firm recommends that the edge of any stormwater infiltration system should maintain a minimum distance of 10 feet away from any existing and proposed foundation system, and 20 feet away from any existing and proposed below-grade retaining wall.

It is recommended that the design team including the structural engineer, waterproofing consultant, plumbing engineer, and landscape architect be consulted in regards to the design and construction of filtration systems. The design and construction of stormwater infiltration systems is not the responsibility of the geotechnical engineer. However, based on the experience of this firm, it is recommended that several aspects of the use of such facilities should be considered by the design and construction team:

- Open infiltration basins have many negative associated issues. Such a design must consider attractive nuisance, impacts to growing vegetation, impacts to air quality and vector control.

- All infiltration devices should be provided with overflow protection. Once the device is full of water, additional water flowing to the device should be diverted to another acceptable disposal area, or disposed offsite in an acceptable manner.

- All connections associated with stormwater infiltration systems should be sealed and water-tight. Water leaking into the subgrade soils can lead to loss of strength, piping, erosion, settlement and/or expansion of the effected earth materials.

- Excavations proposed for the installation of stormwater systems should comply with the "Temporary Excavations" sections of the referenced geotechnical engineering investigation, as well as CalOSHA Regulations where applicable.

DESIGN REVIEW

Engineering of the proposed project should not begin until approval of the geotechnical report by the Building Official is obtained in writing. Significant changes in the geotechnical recommendations may result during the building department review process.

It is recommended that the geotechnical aspects of the project be reviewed by this firm during the design process. This review provides assistance to the design team by providing specific recommendations for particular cases, as well as review of the proposed construction to evaluate whether the intent of the recommendations presented herein are satisfied.

CONSTRUCTION MONITORING

Geotechnical observations and testing during construction are considered to be a continuation of the geotechnical investigation. It is critical that this firm review the geotechnical aspects of the project during the construction process. Compliance with the design concepts, specifications or recommendations during construction requires review by this firm during the course of construction. All foundations should be observed by a representative of this firm prior to placing concrete or steel. Any fill which is placed should be observed, tested, and verified if used for engineered purposes. Please advise Geotechnologies, Inc. at least twenty-four hours prior to any required site visit.

If conditions encountered during construction appear to differ from those disclosed herein, notify Geotechnologies, Inc. immediately so the need for modifications may be considered in a timely manner.

It is the responsibility of the contractor to ensure that all excavations and trenches are properly sloped or shored. All temporary excavations should be cut and maintained in accordance with applicable OSHA rules and regulations.

EXCAVATION CHARACTERISTICS

The exploration performed for this investigation is limited to the geotechnical excavations described. Direct exploration of the entire site would not be economically feasible. The owner, design team and contractor must understand that differing excavation and drilling conditions may be encountered based on boulders, gravel, oversize materials, groundwater and many other conditions. Fill materials, especially when they were placed without benefit of modern grading codes, regularly contain materials which could impede efficient grading and drilling. Excavation and drilling in these areas may require full size equipment and coring capability. The contractor should be familiar with the site and the geologic materials in the vicinity.

CLOSURE AND LIMITATIONS

The purpose of this report is to aid in the design and completion of the described project. Implementation of the advice presented in this report is intended to reduce certain risks associated with construction projects. The professional opinions and geotechnical advice contained in this report are sought because of special skill in engineering and geology and were prepared in accordance with generally accepted geotechnical engineering practice. Geotechnologies, Inc. has a duty to exercise the ordinary skill and competence of members of the engineering profession. Those who hire Geotechnologies, Inc. are not justified in expecting infallibility, but can expect reasonable professional care and competence.

The scope of the geotechnical services provided did not include any environmental site assessment for the presence or absence of organic substances, hazardous/toxic materials in the soil, surface water, groundwater, or atmosphere, or the presence of wetlands.

Proper compaction is necessary to reduce settlement of overlying improvements. Some settlement of compacted fill should be anticipated. Any utilities supported therein should be designed to accept differential settlement. Differential settlement should also be considered at the points of entry to the structure.

GEOTECHNICAL TESTING

Classification and Sampling

The soil is continuously logged by a representative of this firm and classified by visual examination in accordance with the Unified Soil Classification system. The field classification is verified in the laboratory, also in accordance with the Unified Soil Classification System.

Laboratory classification may include visual examination, Atterberg Limit Tests and grain size distribution. The final classification is shown on the excavation logs.

Samples of the geologic materials encountered in the exploratory excavations were collected and transported to the laboratory. Undisturbed samples of soil are obtained at frequent intervals. Unless noted on the excavation logs as an SPT sample, samples acquired while utilizing a hollow-stem auger drill rig are obtained by driving a thin-walled, California Modified Sampler with successive 30-inch drops of a 140-pound hammer. The soil is retained in brass rings of 2.50 inches outside diameter and 1.00 inch in height. The central portion of the samples are stored in close fitting, waterproof containers for transportation to the laboratory. Samples noted on the excavation logs as SPT samples are obtained in accordance with the most recent revision of ASTM D 1586. Samples are retained for 30 days after the date of the geotechnical report.

Moisture and Density Relationships

The field moisture content and dry unit weight are determined for each of the undisturbed soil samples, and the moisture content is determined for SPT samples by the most recent revision of ASTM D 4959 or ASTM D 4643. This information is useful in providing a gross picture of the soil consistency between exploration locations and any local variations. The dry unit weight is determined in pounds per cubic foot and shown on the "Excavation Logs", A-Plates. The field moisture content is determined as a percentage of the dry unit weight.

Direct Shear Testing

Shear tests are performed by the most recent revision of ASTM D 3080 with a strain controlled, direct shear machine manufactured by Soil Test, Inc. or a Direct Shear Apparatus manufactured by GeoMatic, Inc. The rate of deformation is approximately 0.025 inches per minute. Each sample is sheared under varying confining pressures in order to determine the Mohr-Coulomb shear strength parameters of the cohesion intercept and the angle of internal friction. Samples are generally tested in an artificially saturated condition. Depending upon the sample location and future site conditions, samples may be tested at field moisture content. The results are plotted on the "Shear Test Diagram," B-Plates.

The most recent revision of ASTM 3080 limits the particle size to 10 percent of the diameter of the direct shear test specimen. The sheared sample is inspected by the laboratory technician running the test. The inspection is performed by splitting the sample along the sheared plane and observing the soils exposed on both sides. Where oversize particles are observed in the shear plane, the results are discarded and the test run again with a fresh sample.

Consolidation Testing

Settlement predictions of the soil's behavior under load are made on the basis of the consolidation tests using the most recent revision of ASTM D 2435. The consolidation apparatus is designed to receive a single one-inch high ring. Loads are applied in several increments in a geometric progression, and the resulting deformations are recorded at selected time intervals. Porous stones are placed in contact with the top and bottom of each specimen to permit addition and release of pore fluid. Samples are generally tested at increased moisture content to determine the effects of water on the bearing soil. The normal pressure at which the water is added is noted on the drawing. Results are plotted on the "Consolidation Test," C-Plates.

Expansion Index Testing

The expansion tests performed on the remolded samples are in accordance with the Expansion Index testing procedures, as described in the most recent revision of ASTM D4829. The soil sample is compacted into a metal ring at a saturation degree of 50 percent. The ring sample is then placed in a consolidometer, under a vertical confining pressure of 1 lbf/square inch and inundated with distilled water. The deformation of the specimen is recorded for a period of 24 hour or until the rate of deformation becomes less than 0.0002 inches/hour, whichever occurs first. The expansion index, EI, is determined by dividing the difference between final and initial height of the ring sample by the initial height, and multiplied by 1,000. Results are presented in Plate D of this report.

Laboratory Compaction Characteristics

The maximum dry unit weight and optimum moisture content of a soil are determined by use of the most recent revision of ASTM D 1557. A soil at a selected moisture content is placed in five layers into as mold of given dimensions, with each layer compacted by 25 blows of a 10 pound hammer dropped from a distance of 18 inches subjecting the soil to a total compactive effort of about 56,000 pounds per cubic foot. The resulting dry unit weight is determined. The procedure is repeated for a sufficient number of moisture contents to establish a relationship between the dry unit weight and the water content of the soil. The data when plotted, represent a curvilinear relationship known as the compaction curve. The values of optimum moisture content and modified maximum dry unit weight are determined from the compaction curve.
Results are presented in Plate D of this report.

Grain Size Distribution

These tests cover the quantitative determination of the distribution of particle sizes in soils. Sieve analysis is used to determine the grain size distribution of the soil larger than the Number 200 sieve. ASTM D 422-63 (Reapproved 2007) is used to determine particle sizes smaller than the Number 200 sieve. The grain size distributions are plotted on the E-Plate presented in the Appendix of this report.

Atterberg Limits

ASTM D 4318-05 is used to determine the liquid limits, plastic limits, and plasticity index of a soil. These test methods are used to characterize the fine grained fractions of the soil. Results from Atterberg Limits tests are presented in Plate F of this report.

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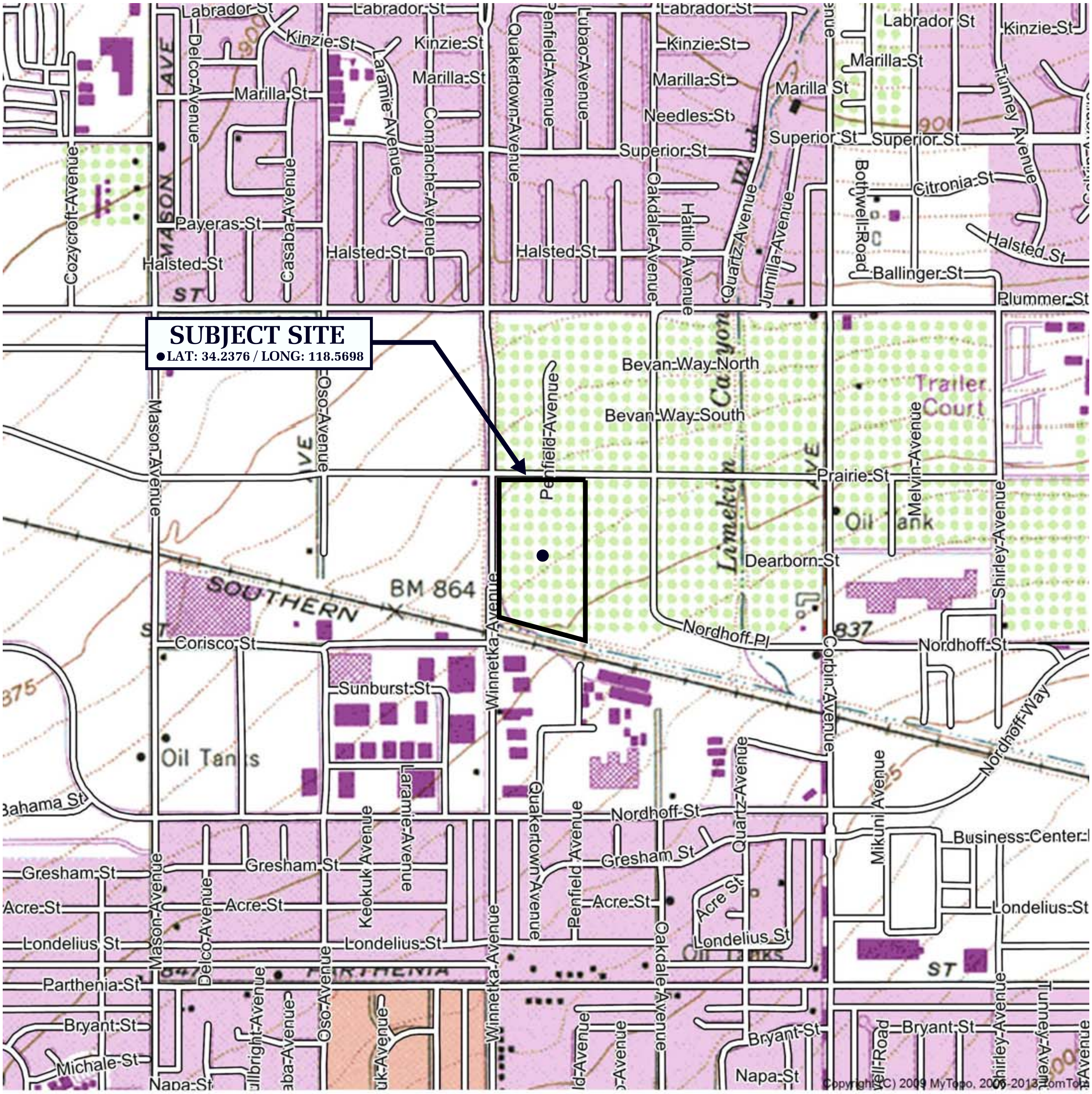


SCALE 1:12000

0.0 0.1 0.2 0.3 0.4 0.5 0.6
MILES

0 1000 2000 3000
FEET

N

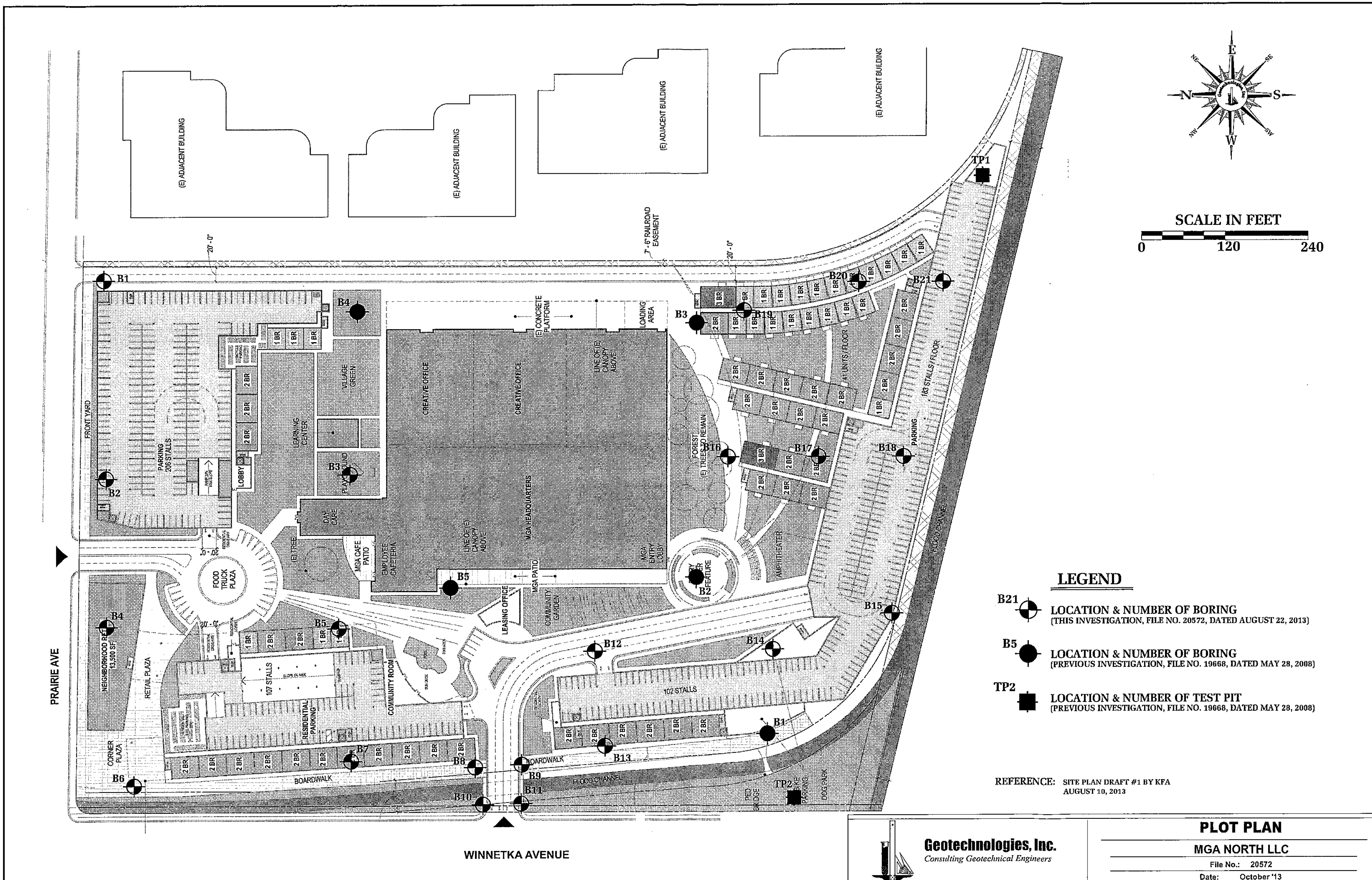
REFERENCE: U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, CANOGA PARK, CA QUADRANGLE

VICINITY MAP

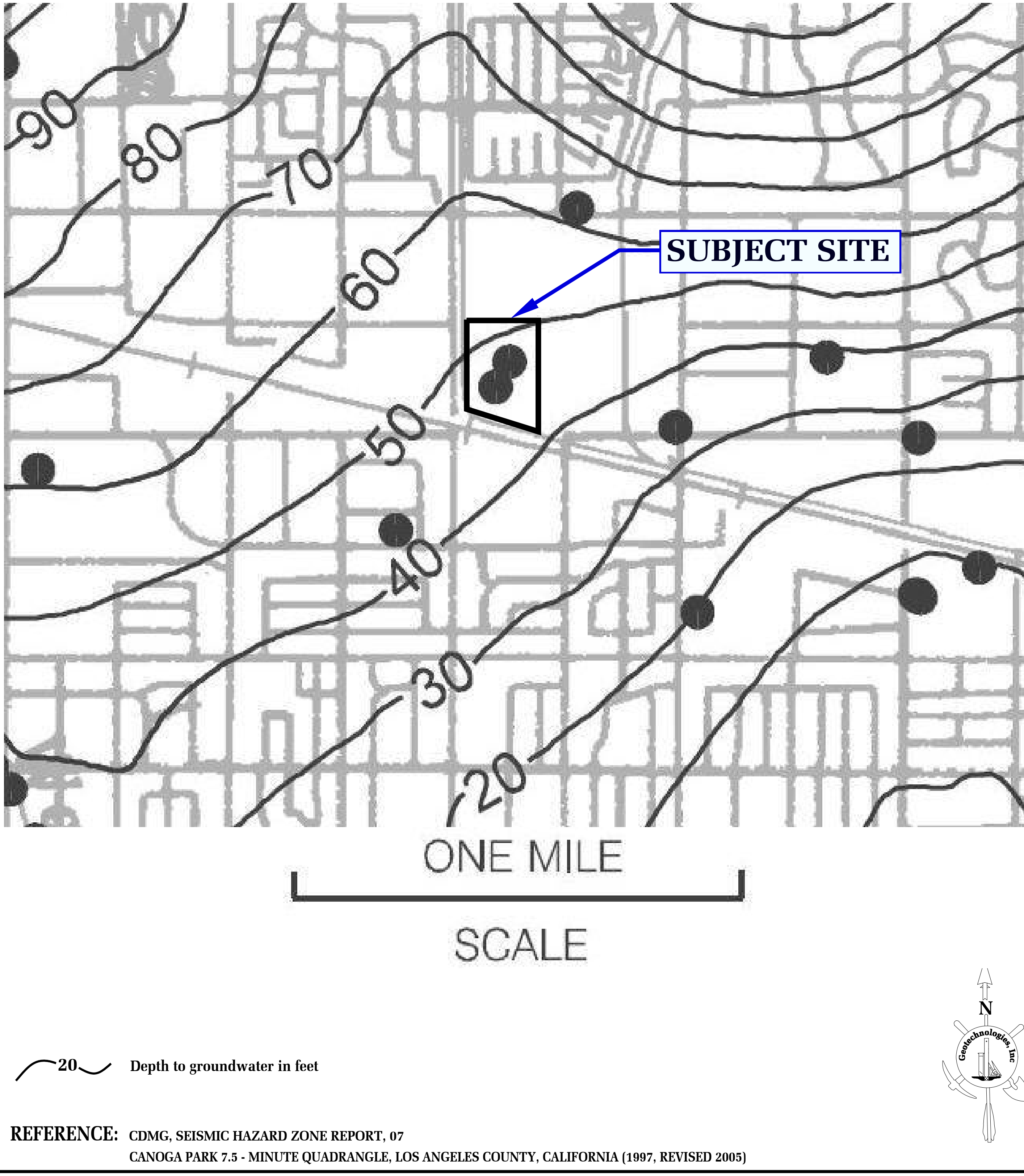
Geotechnologies, Inc.
Consulting Geotechnical Engineers

MGA NORTH, LLC

FILE NO. 20572



(E) ADJACENT BUILDING
(E) ADJACENT BUILDING
(E) ADJACENT BUILDING
(E) ADJACENT BUILDING
SCALE IN FEET
0
120
240
TP1
7'-6" RAILROAD EASEMENT
20'-0"
B1
B2
B3
B4
B5
B6
B7
B8
B9
B10
B11
B12
B13
B14
B15
B16
B17
B18
B19
B20
B21
TP2
FRONT YARD
PARKING 206 STALLS
LOBBY
LEARNING CENTER
VILLAGE GREEN
PLAYGROUND
DAY CARE
CREATIVE OFFICE
CREATIVE OFFICE
LINE OF (E) CANOPY ABOVE
(E) CONCRETE PLATFORM
LOADING AREA
MGA HEADQUARTERS
MGA ENTRY LOBBY
EMPLOYEE CAFETERIA
MGA CAFE PATIO
MGA PATIO
(E) TREE
FOOD TRUCK PLAZA
LEASING OFFICE
COMMUNITY GARDEN
FOREST (E) TREES TO REMAIN
AMPHITHEATER
WATER FEATURE
141 UNITS / FLOOR
183 STALLS / FLOOR
PARKING
FLOOD CHANNEL
NEIGHBORHOOD RETAIL 13,500 SF
RETAIL PLAZA
CORNER PLAZA
107 STALLS
RESIDENTIAL PARKING
COMMUNITY ROOM
102 STALLS
BOARDWALK
BOARDWALK
FLOOD CHANNEL
PED BRIDGE
BIKE PARKING
DOG PARK
PRAIRIE AVE
WINNETKA AVENUE
LEGEND
B21 LOCATION & NUMBER OF BORING (THIS INVESTIGATION, FILE NO. 20572, DATED AUGUST 22, 2013)
B5 LOCATION & NUMBER OF BORING (PREVIOUS INVESTIGATION, FILE NO. 19668, DATED MAY 28, 2008)
TP2 LOCATION & NUMBER OF TEST PIT (PREVIOUS INVESTIGATION, FILE NO. 19668, DATED MAY 28, 2008)
REFERENCE: SITE PLAN DRAFT #1 BY KFA AUGUST 10, 2013
Geotechnologies, Inc.
Consulting Geotechnical Engineers
PLOT PLAN
MGA NORTH LLC
File No.: 20572
Date: October '13



20 Depth to groundwater in feet

REFERENCE: CDMG, SEISMIC HAZARD ZONE REPORT, 07
CANOGA PARK 7.5 - MINUTE QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA (1997, REVISED 2005)

HISTORICALLY HIGHEST GROUNDWATER LEVELS

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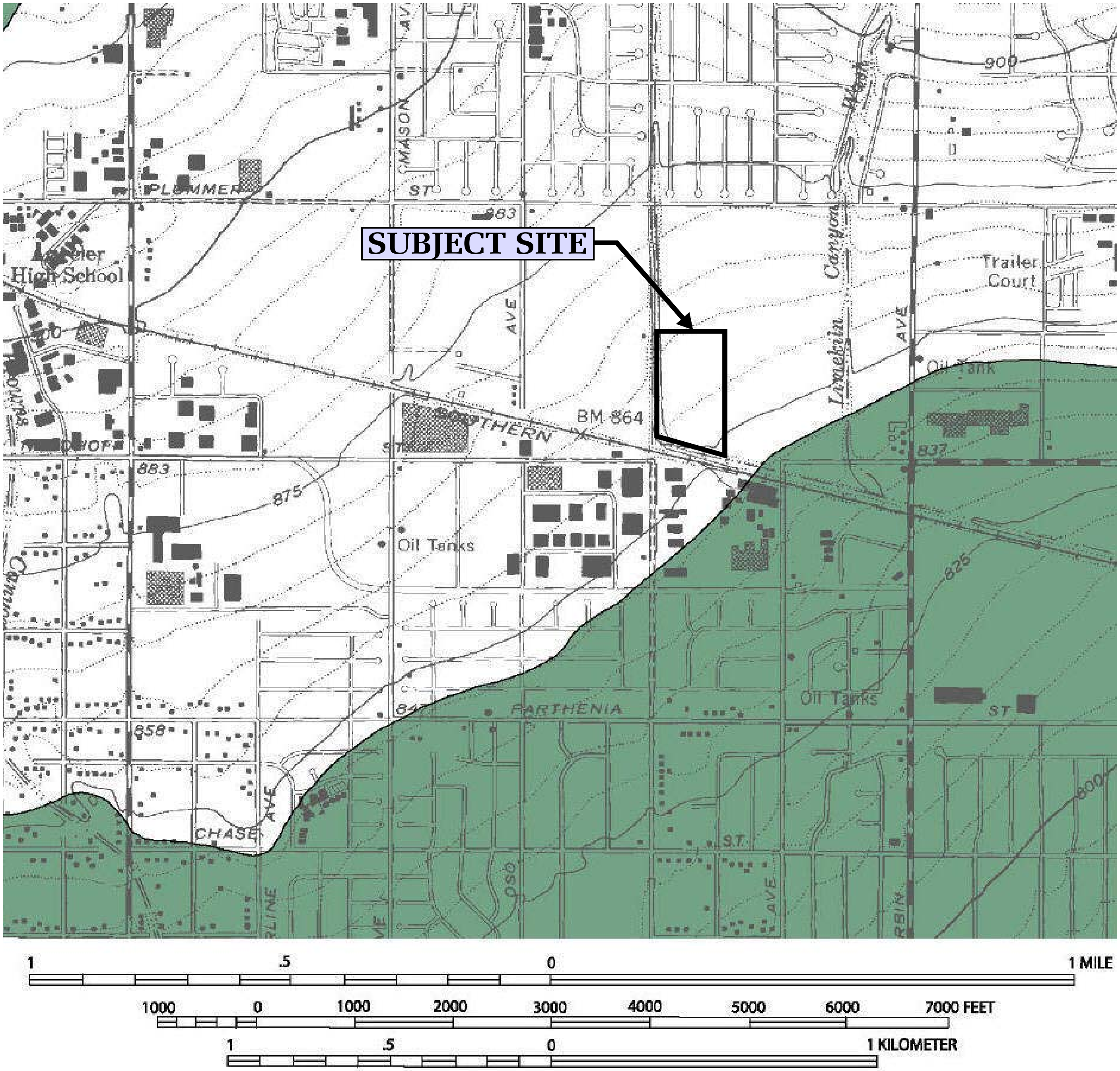
FILE No. 20572



LIQUEFACTION AREA

REFERENCE: SEISMIC HAZARD ZONES,CANOGA PARK QUADRANGLE OFFICIAL MAP (CDMG, 1998)

SEISMIC HAZARD ZONE MAP

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Consulting Geotechnical Engineers

MGA NORTH, LLC

FILE NO. 20572

BORING LOG NUMBER 1

MGA North, LLC **Date: 07/08/13**

File No. 20572 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		3-inch Asphalt over 2-inch Base
				-		
				1 --		FILL: Sandy Silt, dark brown, moist, stiff
				-		
				2 --		
2.5	21	10.3	101.4	-		
				3 --	ML	ALLUVIUM: Sandy Silt, dark to yellowish brown, slightly moist, stiff
				-		
				4 --		
				-		
5	23	8.5	98.2	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	36	11.4	94.7	-		
				8 --		
				-		
				9 --		
				-		
10	34	8.7	98.0	10 --		
				-	SM/ML	Silty Sand to Sandy Silt, yellowish brown, slightly moist, medium dense, fine grained, stiff
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	39	3.0	104.7	15 --		
				-	SM/SP	Silty Sand to Sand, yellowish brown, slightly moist, medium dense, fine grained
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-	SP	Sand, yellowish brown, slightly moist, medium dense, fine grained
20	33	2.0	103.9	20 --		
				-		Total depth: 20 feet
				21 --		No Water
				-		Fill to 2½ feet
				22 --		
				-		
				23 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual
				24 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				25 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted

BORING LOG NUMBER 2

MGA North, LLC **Date: 07/09/13**

File No. 20572 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		7-inch Asphalt, No Base
				-		
				1 --		FILL: Sandy Silt, dark to yellowish brown, moist, stiff
				-		
				2 --		
2.5	35	15.0	108.3	-		
				3 --		
				-	ML	ALLUVIUM: Sandy Silt, yellowish brown, moist, stiff
				4 --		
				-		
5	75	12.5	121.4	5 --		
				-	ML/SM	Sandy Silt to Silty Sand, yellowish brown, moist, stiff, medium dense to dense, fine grained
				6 --		
				-		
				7 --		
7.5	75	4.9	116.7	-		
				8 --	SM/SW	Silty Sand to Gravelly Sand, dark to yellowish brown, moist, very dense, fine to coarse grained
				-		
				9 --		
				-		
10	54	3.9	117.6	10 --		
				-	SP/SW	Sand to Gravelly Sand, yellowish brown, moist, dense, fine to coarse grained
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	36	5.7	99.6	15 --		
				-	SM/SP	Silty Sand to Sand, yellowish brown, moist, medium dense, fine grained
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	63	3.9	106.6	20 --		
				-		Total depth: 20 feet
				21 --		No Water
				-		Fill to 3 feet
				22 --		
				-		
				23 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual
				24 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				25 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted

BORING LOG NUMBER 3

MGA North, LLC **Date: 07/08/13**

File No. 20572 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
				0 --		FILL: Sandy to Clayey Silt, dark to yellowish brown, moist, stiff
				-		
				1 --		
				-		
				2 --		
2.5	19	28.0	95.9	-		
				3 --	ML	ALLUVIUM: Sandy Silt, dark to yellowish brown, moist, stiff
				-		
				4 --		
				-		
5	21	3.7	104.6	5 --		
				-	SP/SW	Sand to Gravelly Sand, dark to grayish brown, moist, medium
				6 --		dense, fine to coarse grained
				-		
				7 --		
7.5	18	3.6	104.6	-		
				8 --		
				-		
				9 --		
				-		
10	14	27.7	89.7	10 --		
				-	ML	Sandy to Clayey Silt, dark brown, moist, stiff
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	16	26.0	92.9	15 --		
				-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-	SM	Silty Sand, yellowish brown, moist, medium dense, fine grained
20	33	13.7	107.0	20 --		
				-		Total depth: 20 feet
				21 --		No Water
				-		Fill to 2½ feet
				22 --		
				-		
				23 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual
				24 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				25 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted

BORING LOG NUMBER 4

MGA North, LLC **Date: 07/09/13**

File No. 20572 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		3-inch Asphalt over 7-inch Base
				-		
				1 --		FILL: Sandy Silt, dark brown, moist, stiff
				-		
				2 --		
2.5	58	12.0	124.4	-		
				3 --		
				-	SM/ML	ALLUVIUM; Silty Sand to Sandy Silt, dark to yellowish brown,
				4 --		moist, medium dense to dense, fine grained, stiff
				-		
5	48	13.6	117.8	5 --		
				-	ML	Sandy Silt, yellowish brown, moist, stiff
				6 --		
				-		
				7 --		
7.5	34	11.3	109.0	-		
				8 --	SM/ML	Silty Sand to Sandy Silt, yellowish brown, moist, medium dense,
				-		fine grained, stiff
				9 --		
				-		
10	22	11.9	107.4	10 --		
				-	SM	Silty Sand, yellowish brown, moist, medium dense, fine grained
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	26	13.9	98.6	15 --		
				-	ML	Sandy Silt, yellowish brown, moist, stiff
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	31	13.2	93.5	20 --		
				-		Total depth: 20 feet
				21 --		No Water
				-		Fill to 3 feet
				22 --		
				-		
				23 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual
				24 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				25 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted

BORING LOG NUMBER 5

MGA North, LLC **Date: 07/09/13**

File No. 20572 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		3-inch Asphalt over 3-inch Base
				-		
				1 --		FILL: Sandy Silt, dark and grayish brown, moist, stiff
				-		
				2 --		
2.5	29	16.6	111.5	-		
				3 --		
				-		
				4 --		
				-		
5	40	14.8	117.1	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	61	11.8	124.2	-		
				8 --	ML/SM	ALLUVIUM: Sandy Silt to Silty Sand, dark to yellowish brown, moist, stiff, medium dense, fine grained
				-		
				9 --		
				-		
10	47	15.8	116.4	10 --		
				-	ML	Sandy Silt, yellowish brown, moist, stiff
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	30	20.1	107.0	15 --		
				-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		Silty Sand to Sandy Silt, yellowish brown, moist, medium dense, fine grained, stiff
				-	SM/ML	
20	31	12.0	106.5	20 --		
				-		Total depth: 20 feet
				21 --		No Water
				-		Fill to 7½ feet
				22 --		
				-		
				23 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual
				24 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				25 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted

BORING LOG NUMBER 6

MGA North, LLC | **Date: 07/09/13**

File No. 20572 | **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		3-inch Asphalt over 3-inch Base
				-		
				1 --		FILL: Sandy Silt, dark to yellowish brown, moist, stiff
				-		
				2 --		
2.5	85	8.4	124.3	-		
				3 --	SM/ML	ALLUVIUM: Sandy Silt to Silty Sand, yellowish brown, moist, stiff, medium, dense, fine grained
				-		
				4 --		
				-		
5	26	9.8	SPT	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	86	13.4	120.3	-		
				8 --		
				-		
				9 --		
				-		
10	14	13.1	SPT	10 --		
				-		
				11 --		
				-		
				12 --		
12.5	40	13.9	109.6	-		
				13 --		
				-		
				14 --		
				-		
15	8	15.4	SPT	15 --		
				-	ML	Sandy Silt, yellowish brown, moist, stiff
				16 --		
				-		
				17 --		
17.5	33	16.2	107.2	-		
				18 --		
				-		
				19 --		
				-		
20	11	16.4	SPT	20 --		
				-		
				21 --		
				-		
				22 --		
22.5	54	13.1	113.7	-		
				23 --	SM/ML	Silty Sand to Sandy Silt, yellowish brown, moist, medium dense to dense, fine grained, stiff
				-		
				24 --		
				-		
25	11	8.5	SPT	25 --		
				-		

BORING LOG NUMBER 6

MGA North, LLC

File No. 20572

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual
				27 --		
27.5	46	12.3	115.2	-		
				28 --		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				-		
				29 --		
				-		
30	14	16.2	SPT	30 --		SPT=Standard Penetration Test
				-		
				31 --		
				-		
				32 --		
32.5	70	15.5	117.0	-		
				33 --	ML	Sandy to Clayey Silt, dark brown, moist, very stiff
				-		
				34 --		
				-		
35	11	13.0	SPT	35 --		
				-		
				36 --		
				-		
				37 --		
37.5	54	10.9	116.9	-		
				38 --	ML/SM	Sandy Silt to Silty Sand, dark and medium brown, moist, stiff, dense, fine grained
				-		
				39 --		
				-		
40	14	13.1	SPT	40 --		
				-	CH	Sandy Clay, yellowish brown, moist, stiff
				41 --		
				-		
				42 --		
42.5	67	15.6	96.8	-		
				43 --	SM/ML	Silty Sand to Sandy Silt, dark to yellowish brown, moist, dense, fine grained, stiff, minor rock fragments
				-		
				44 --		
				-		
45	31	11.1	SPT	45 --		
				-		
				46 --		
				-		
				47 --		
47.5	88	4.5	109.7	-		
				48 --	SP/SW	Sand to Gravelly Sand, yellowish brown, moist, very dense, fine to coarse grained
				-		
				49 --		
				-		
50	65	6.5	SPT	50 --		
				-		Total depth: 50 feet No Water Fill to 2½ feet

BORING LOG NUMBER 7

MGA North, LLC **Date: 07/09/13**

File No. 20572 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		3-inch Asphalt over 2-inch Base
				-		
				1 --		FILL: Silty Sand to Sandy Silt, dark and yellowish brown, moist, medium dense or stiff, fine grained
				-		
				2 --		
2.5	26	4.9	109.4	-		
				3 --	SM/ML	ALLUVIUM: Silty Sand to Sandy Silt, yellowish brown, slightly moist, medium dense, fine grained, stiff
				-		
				4 --		
				-		
5	22	10.2	99.4	5 --		
				-	ML	Sandy Silt, yellowish brown, moist, stiff
				6 --		
				-		
				7 --		
7.5	23	11.6	99.8	-		
				8 --		
				-		
				9 --		
				-		
10	32	13.6	103.0	10 --		
				-		
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	31	17.2	101.9	15 --		
				-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		Sandy Silt to Silty Sand, yellowish brown, moist, very dense, fine grained
				-	SM/ML	
20	82	8.7	123.1	20 --		
				-		Total depth: 20 feet No Water Fill to 2½ feet
				21 --		
				-		
				22 --		
				-		
				23 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual
				-		
				24 --		
				-		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				25 --		
				-		

BORING LOG NUMBER 8

MGA North, LLC **Date: 07/10/13**

File No. 20572 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
				0 --		FILL: Sandy Silt, yellowish brown, slightly moist, stiff
				-		
				1 --		
				-		
				2 --		
2.5	24	6.7	101.8	-		
				3 --	ML	ALLUVIUM: Sandy Silt, yellowish brown, moist, stiff
				-		
				4 --		
				-		
5	22	6.4	92.4	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	35	4.6	107.2	-		
				8 --	SM/ML	Silty Sand to Sandy Silt, yellowish brown, moist, medium dense, fine grained, stiff
				-		
				9 --		
				-		
10	22	6.7	102.1	10 --		
				-	ML	Sandy Silt, yellowish brown, moist, stiff
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	34	8.5	102.0	15 --		
				-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		Silty Sand to Sandy Silt, yellowish brown, moist, stiff, medium
				-	SM/ML	dense, fine grained
20	44	10.5	104.1	20 --		
				-		Total depth: 20 feet
				21 --		No Water
				-		Fill to 2½ feet
				22 --		
				-		
				23 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual
				24 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				25 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted

BORING LOG NUMBER 9

MGA North, LLC | **Date: 07/10/13**

File No. 20572 | **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
				0 --		FILL: Sandy Silt, yellowish brown, moist, stiff
				-		
				1 --		
				-		
				2 --		
2.5	22	2.9	111.9	-		
				3 --	SM/ML	ALLUVIUM: Sandy Silt to Silty Sand, yellowish brown, moist,
				-		stiff, medium dense, fine grained
				4 --		
				-		
5	23	6.3	94.3	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	28	4.9	107.6	-		
				8 --		
				-		
				9 --		
				-		
10	19	6.7	100.3	10 --		
				-	ML	Sandy Silt, yellowish brown, moist, stiff
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	33	9.7	103.6	15 --		
				-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	38	10.2	113.0	20 --		
				-		Total depth: 20 feet
				21 --		No Water
				-		Fill to 2½ feet
				22 --		
				-		
				23 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual
				24 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				25 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted

BORING LOG NUMBER 10

MGA North, LLC | **Date: 07/10/13**

File No. 20572 | **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
				0 --		FILL: Sandy Silt, yellowish brown, moist, stiff
				-		
				1 --		
				-		
				2 --		
2.5	26	4.1	111.1	-		
				3 --	ML	ALLUVIUM: Sandy Silt, yellowish brown, moist, stiff
				-		
				4 --		
				-		
5	30	4.4	101.9	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	27	6.1	101.1	-		
				8 --	SM/ML	Sandy Silt to Silty Sand, yellowish brown, moist, stiff, medium
				-		dense, fine grained
				9 --		
				-		
10	20	6.9	99.6	10 --		
				-		
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	46	8.6	100.9	15 --		
				-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	31	7.1	104.9	20 --		
				-		Total depth: 20 feet
				21 --		No Water
				-		Fill to 2½ feet
				22 --		
				-		
				23 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual
				24 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				25 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted

BORING LOG NUMBER 11

MGA North, LLC **Date: 07/10/13**

File No. 20572 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
				0 --		FILL: Sandy Silt, yellowish brown, moist, stiff
				-		
				1 --		
				-		
				2 --		
2.5	53	7.1	104.9	-		
				3 --	ML	ALLUVIUM: Sandy Silt, yellowish brown, moist, stiff
				-		
				4 --		
				-		
5	45	7.5	103.8	5 --		
				-	ML/SM	Sandy Silt to Silty Sand, yellowish brown, moist, stiff, medium dense, fine grained
				6 --		
				-		
				7 --		
7.5	33	2.3	104.6	-		
				8 --	SM	Silty Sand, yellowish brown, slightly moist, medium dense, fine grained
				-		
				9 --		
				-		
10	22	5.5	92.7	10 --		
				-	SM/ML	Silty Sand to Sandy Silt, yellowish brown, moist, medium dense, fine grained, stiff
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	43	7.9	100.9	15 --		
				-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	32	8.4	101.4	20 --		
				-		Total depth: 20 feet
				21 --		No Water
				-		Fill to 2½ feet
				22 --		
				-		
				23 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual
				24 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				25 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted

BORING LOG NUMBER 12

MGA North, LLC | **Date: 07/09/13**

File No. 20572 | **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Concrete
				0 --		9-inch Concrete, No Base
				-		
				1 --		FILL: Silty Sand, yellowish brown, moist, medium dense, fine
				-		grained with gravel
				2 --		
				-		Silty Sand with Gravel, yellowish brown, moist, medium dense to
3	64	4.8	126.6	3 --		dense, fine grained
				-		
				4 --		
				-		
5	71	4.8	130.2	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	69	8.1	127.7	-		
				8 --		
				-		
				9 --		
				-		
10	85	6.4	132.1	10 --		
				-		
				11 --		
				-		
				12 --		
12.5	78	11.8	99.9	-		
				13 --	SM/SP	ALLUVIUM: Silty Sand to Sand, yellow to grayish brown, moist,
				-		very dense, fine to medium grained
				14 --		
				-		
15	29	8.7	107.0	15 --		
				-	SP	Sand, gray, moist, medium dense, fine grained
				16 --		
				-		
				17 --		
17.5	20	17.0	109.5	-		
				18 --	ML	Sandy Silt, yellowish brown, moist, stiff
				-		
				19 --		Silty Sand to Sandy Silt, yellowish brown, moist, medium dense,
				-	SM/ML	fine grained, stiff
20	26	14.4	102.2	20 --		
				-		Total depth: 20 feet
				21 --		No Water
				-		Fill to 2 feet
				22 --		
				-		
				23 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual
				24 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				25 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted

BORING LOG NUMBER 13

MGA North, LLC **Date: 07/09/13**

File No. 20572 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Concrete
				0 --		8-inch Concrete over 3-inch Base
				-		
				1 --		
				-		FILL: Sandy Silt, dark to yellowish brown, moist, stiff
				2 --		
2.5	39	7.7	108.6	-		
				3 --	SM/ML	ALLUVIUM: Silty Sand to Sandy Silt, dark to yellowish brown, moist, medium dense, fine grained
				-		
				4 --		
				-		
5	33	8.9	105.1	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	34	10.6	96.9	-		
				8 --		
				-		
				9 --		
				-		
10	30	8.0	104.3	10 --		
				-		
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	29	9.4	109.1	15 --		
				-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	39	7.9	116.0	20 --		
				-		Total depth: 20 feet
				21 --		No Water
				-		Fill to 2½ feet
				22 --		
				-		
				23 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual
				24 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				25 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted

BORING LOG NUMBER 14

MGA North, LLC **Date: 07/09/13**

File No. 20572 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
				0 --		FILL: Sandy Silt to Silty Sand, dark to yellowish brown, moist, stiff, medium dense, fine grained
				-		
				1 --		
				-		
				2 --		
2.5	70	7.7	119.2	-		
				3 --	SM/ML	ALLUVIUM: Sandy Silt to Silty Sand, yellowish brown, moist, very stiff, very dense, fine grained
				-		
				4 --		
				-		
5	100/7"	3.0	111.5	5 --		
				-		some cobbles
				6 --		
				-		
				7 --		
7.5	80	9.2	124.6	-		Sandy Silt to Silty Sand, yellowish brown, moist, very stiff, dense, fine grained, minor gravel
				8 --		
				-		
				9 --		
				-		
10	64	11.5	121.4	10 --		
	50/5"			-	ML	Sandy Silt, dark brown, moist, very stiff
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	35	6.9	120.0	15 --		
				-	ML/SM	Sandy Silt to Silty Sand, dark to yellowish brown, moist, medium dense, fine grained, stiff
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	26	10.9	112.4	20 --		
				-		Total depth: 20 feet
				21 --		No Water
				-		Fill to 2½ feet
				22 --		
				-		
				23 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual
				24 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				25 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted

BORING LOG NUMBER 15

MGA North, LLC | **Date: 07/10/13**

File No. 20572 | **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
				0 --		FILL: Sandy Silt, yellowish brown, moist, stiff
				-		
				1 --		
				-		
				2 --		
2.5	76	7.6	119.9	-		
	50/3"			3 --	ML	ALLUVIUM: Sandy Silt, yellowish brown, slightly moist, very stiff
				-		
				4 --		
				-		
5	52	6.2	109.9	5 --		
				-	ML/SM	Silty Sand to Sandy Silt, yellowish brown, moist, medium dense to dense, fine grained, stiff
				6 --		
				-		
				7 --		
7.5	30	6.7	102.4	-		
				8 --	ML	Sandy Silt, yellowish brown, moist, stiff
				-		
				9 --		
				-		
10	13	6.3	90.4	10 --		
				-		
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	37	8.6	97.4	15 --		
				-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		Silty Sand to Sandy Silt, yellowish brown, moist, medium dense,
				-	SM/ML	fine grained, stiff
20	42	6.7	102.6	20 --		
				-		Total depth: 20 feet
				21 --		No Water
				-		Fill to 2½ feet
				22 --		
				-		
				23 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual
				24 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				25 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted

BORING LOG NUMBER 16

MGA North, LLC — **Date: 07/08/13**

File No. 20572 — **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
				0 --		FILL: Sandy Silt, yellowish brown, slightly moist, stiff
				-		
				1 --		
				-		
				2 --		
2.5	58	5.8	115.6	-		
				3 --	ML/SM	ALLUVIUM: Sandy Silt to Silty Sand, yellowish brown, slightly moist, dense, fine grained
				-		
				4 --		
				-		
5	83	6.4	117.2	5 --		
				-	SM	Silty Sand, dark to yellowish brown, slightly moist, very dense, fine grained
				6 --		
				-		
				7 --		
7.5	37 50/5"	7.8	118.2	-		
				8 --		
				-		
				9 --		
				-		
10	37 50/5"	6.5	119.2	10 --		
				-		
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	29	9.3	99.7	15 --		
				-	ML	Sandy Silt, yellowish brown, slightly moist, stiff
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	42	11.0	94.6	20 --		
				-		Total depth: 20 feet
				21 --		No Water
				-		Fill to 2½ feet
				22 --		
				-		
				23 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual
				-		
				24 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				25 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted

BORING LOG NUMBER 17

MGA North, LLC | **Date: 07/08/13**

File No. 20572 | **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
				0 --		FILL: Sandy Silt, yellowish brown, moist, stiff
				-		
				1 --		
				-		
				2 --		
2.5	38	5.3	115.9	-		
				3 --	ML	ALLUVIUM: Sandy Silt, yellowish brown, moist, stiff
				-		
				4 --		
				-		
5	77	7.8	110.9	5 --		
				-	ML/SM	Sandy Silt to Silty Sand, dark and yellowish brown, moist, very dense, fine grained
				6 --		
				-		
				7 --		
7.5	87	7.1	117.9	-		
				8 --	ML	Sandy Silt, yellowish brown, moist, very stiff
				-		
				9 --		
				-		
10	62	8.3	118.5	10 --		
	50/5"			-		
				11 --		
				-		
				12 --		
12.5	100/8"	6.7	118.4	-		
				13 --	SM	Silty Sand, dark brown, moist, very dense, fine grained
				-		
				14 --		
				-		
15	76	8.6	119.1	15 --		
				-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	100/12"	2.8	119.1	20 --		
				-		Total depth: 20 feet
				21 --		No Water
				-		Fill to 2½ feet
				22 --		
				-		
				23 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual
				24 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				25 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted

BORING LOG NUMBER 18

MGA North, LLC **Date: 07/08/13**

File No. 20572 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
				0 --		FILL: Sandy Silt, yellowish brown, slightly moist, stiff
				-		
				1 --		
				-		
				2 --		
2.5	41	6.7	107.7	-		
				3 --	ML	ALLUVIUM: Sandy Silt, yellowish brown, slightly moist, stiff
				-		
				4 --		
				-		
5	29	4.9	104.0	5 --		
				-	ML/SP	Sandy Silt to Sand, yellowish brown, slightly moist, stiff, medium dense, fine grained
				6 --		
				-		
				7 --		
7.5	77 50/5"	4.8	120.0	-		
				8 --	SM	Silty Sand, yellowish brown, slightly moist, vey dense, fine grained
				-		
				9 --		
				-		
10	80	4.8	113.0	10 --		
				-		
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	38	11.2	94.9	15 --		
				-	ML	Sandy Silt, yellowish brown, slightly moist, stiff
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	45	8.4	107.7	20 --		
				-		Total depth: 20 feet
				21 --		No Water
				-		Fill to 2½ feet
				22 --		
				-		
				23 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual
				24 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				25 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted

BORING LOG NUMBER 19

MGA North, LLC **Date: 07/08/13**

File No. 20572 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Concrete
				0 --		8-inch Concrete over 4-inch Base
				1 --		FILL: Sandy Silt to Silty Sand, dark and medium brown, moist, stiff, medium dense, fine grained
				2 --		
2.5	33	12.7	113.4	-		
				3 --		
				4 --		
5	13	9.6	SPT	5 --		
				6 --	ML	ALLUVIUM; Sandy Silt, yellowish brown, moist, stiff
				7 --		
7.5	31	7.3	117.0	-		
				8 --	SM	Silty Sand, yellowish brown, slightly moist, medium dense, fine grained, minor gravel
				9 --		
10	17	3.3	SPT	10 --	SM/SP	Silty Sand to Sand, yellowish brown, slightly moist, medium dense, fine grained
				11 --		
				12 --		
12.5	48	3.5	111.9	-		
				13 --	SP	Sand, grayish brown, slightly moist, medium dense, fine to medium grained, minor gravel
				14 --		
15	27	2.8	SPT	15 --		
				16 --		
				17 --		
17.5	26	13.1	92.8	-		
				18 --	SM/ML	Silty Sand to Sandy Silt, dark to yellowish brown, moist, medium dense, fine grained, stiff
				19 --		
20	14	16.4	SPT	20 --	ML	Sandy Silt, yellowish brown, moist, stiff
				21 --		
				22 --		
22.5	37	14.8	103.3	-		
				23 --		
				24 --		
25	13	13.8	SPT	25 --		

BORING LOG NUMBER 19

MGA North, LLC

File No. 20572

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual
				27 --		
27.5	45	10.1	104.6	-		Used 8-inch diameter Hollow-Stem Auger
				28 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted
				29 --		
				-		SPT=Standard Penetration Test
30	19	9.1	SPT	30 --		
				-		
				31 --		
				-		
				32 --		
32.5	61	2.7	109.2	-		
				33 --		
				-		
				34 --		
				-		
35	14	7.8	SPT	35 --		
				-	SM	Silty Sand, dark brown, moist, medium dense, fine grained
				36 --		
				-		
				37 --		
37.5	79	4.7	109.3	-		
				38 --	SM/SP	Silty Sand to Sand, dark to medium brown, moist, dense, fine
				-		grained
				39 --		
				-		
40	20	16.3	SPT	40 --		
				-	ML	Clayey Silt, dark brown, moist, stiff
				41 --		
				-		
				42 --		
42.5	57	16.6	114.2	-		
				43 --		
				-		
				44 --		
				-		
45	17	13.0	SPT	45 --		
				-	CH	Sandy to Silty Clay, dark brown, moist, stiff
				46 --		
				-		
				47 --		
47.5	41	15.6	116.2	-		
				48 --		
				-		
				49 --		
				-		
50	21	18.8	SPT	50 --		
				-		Total depth: 50 feet
						No Water
						Fill to 6 feet

BORING LOG NUMBER 20

MGA North, LLC **Date: 07/08/13**

File No. 20572 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
				0 --		FILL: Sandy Silt, dark to yellowish brown, moist, stiff
				-		
				1 --		
				-		
				2 --		
2.5	22	6.3	117.0	-		
				3 --	ML/SM	ALLUVIUM: Sandy Silt to Silty Sand, dark to yellowish brown, moist, stiff, medium dense, fine grained
				-		
				4 --		
				-		
5	12	3.1	102.5	5 --		
				-	SP/SM	Sand to Silty Sand, yellowish brown, slightly moist, medium dense, fine grained, stiff
				6 --		
				-		
				7 --		
7.5	20	7.5	90.3	-		
				8 --	ML	Sandy Silt, yellowish brown, moist, stiff
				-		
				9 --		
				-		
10	24	9.1	95.5	10 --		
				-		
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	34	12.9	92.7	15 --		
				-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	33	9.9	97.6	20 --		
				-		Total depth: 20 feet
				21 --		No Water
				-		Fill to 2½ feet
				22 --		
				-		
				23 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual
				24 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				25 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted

BORING LOG NUMBER 21

MGA North, LLC | **Date: 07/08/13**

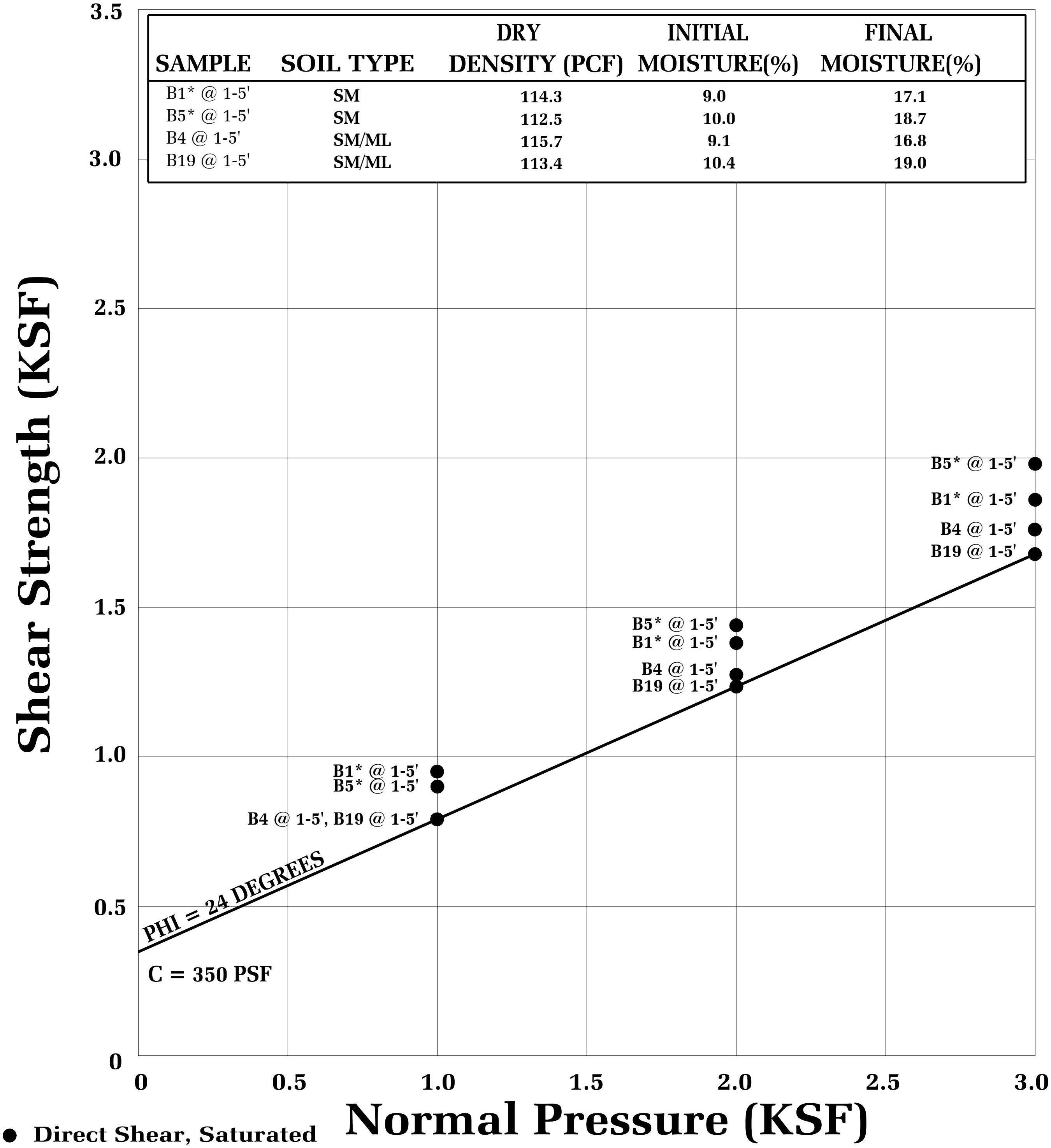
File No. 20572 | **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
				0 --		FILL: Sandy Silt, dark to yellowish brown, moist, stiff
				-		
				1 --		
				-		
				2 --		
2.5	24	6.1	100.2	-	ML	ALLUVIUM: Sandy Silt, dark to yellowish brown, moist, stiff
				3 --		
				-		
				4 --		
				-		
5	27	6.3	103.5	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	32	8.9	99.7	-		
				8 --		
				-		
				9 --		
				-		
10	33	9.7	102.6	10 --		
				-		
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	40	11.5	98.1	15 --		
				-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	46	5.9	114.3	20 --		
				-		Total depth: 20 feet
				21 --		No Water
				-		Fill to 2 feet
				22 --		
				-		
				23 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual
				24 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				25 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted

BULK SAMPLE REMOLDED TO 90 PERCENT OF THE MAXIMUM LABORATORY DENSITY

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B1* @ 1-5'	SM	114.3	9.0	17.1
B5* @ 1-5'	SM	112.5	10.0	18.7
B4 @ 1-5'	SM/ML	115.7	9.1	16.8
B19 @ 1-5'	SM/ML	113.4	10.4	19.0



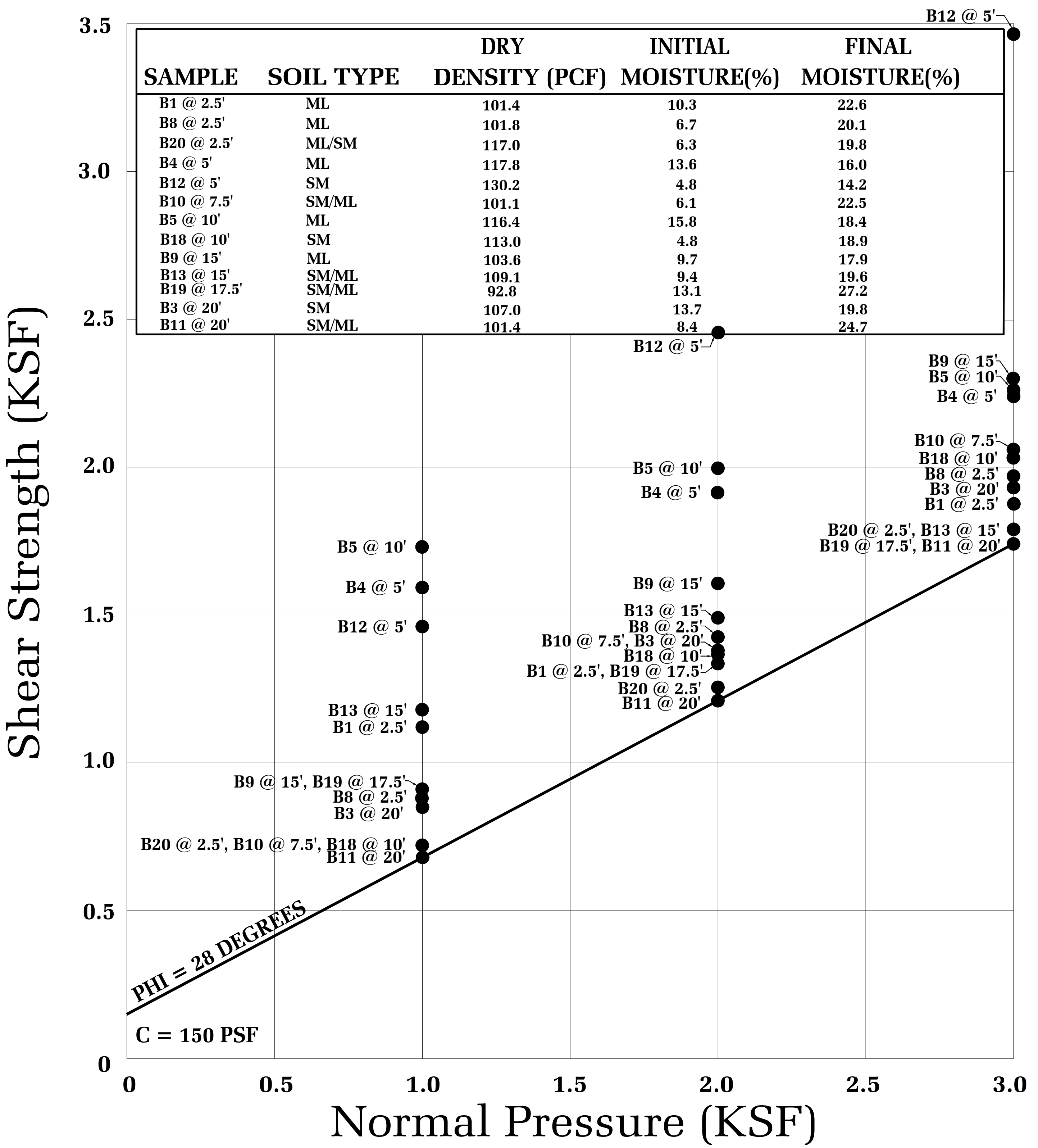
*BORING DRILLED DURING A PREVIOUS SITE EXPLORATION (GEOTECHNOLOGIES, INC., 2008, FILE No. 19668))

SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

MGA NORTH, LLC

FILE NO. 20572 | PLATE: B-1



SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B1 @ 2.5'	ML	101.4	10.3	22.6
B8 @ 2.5'	ML	101.8	6.7	20.1
B20 @ 2.5'	ML/SM	117.0	6.3	19.8
B4 @ 5'	ML	117.8	13.6	16.0
B12 @ 5'	SM	130.2	4.8	14.2
B10 @ 7.5'	SM/ML	101.1	6.1	22.5
B5 @ 10'	ML	116.4	15.8	18.4
B18 @ 10'	SM	113.0	4.8	18.9
B9 @ 15'	ML	103.6	9.7	17.9
B13 @ 15'	SM/ML	109.1	9.4	19.6
B19 @ 17.5'	SM/ML	92.8	13.1	27.2
B3 @ 20'	SM	107.0	13.7	19.8
B11 @ 20'	SM/ML	101.4	8.4	24.7

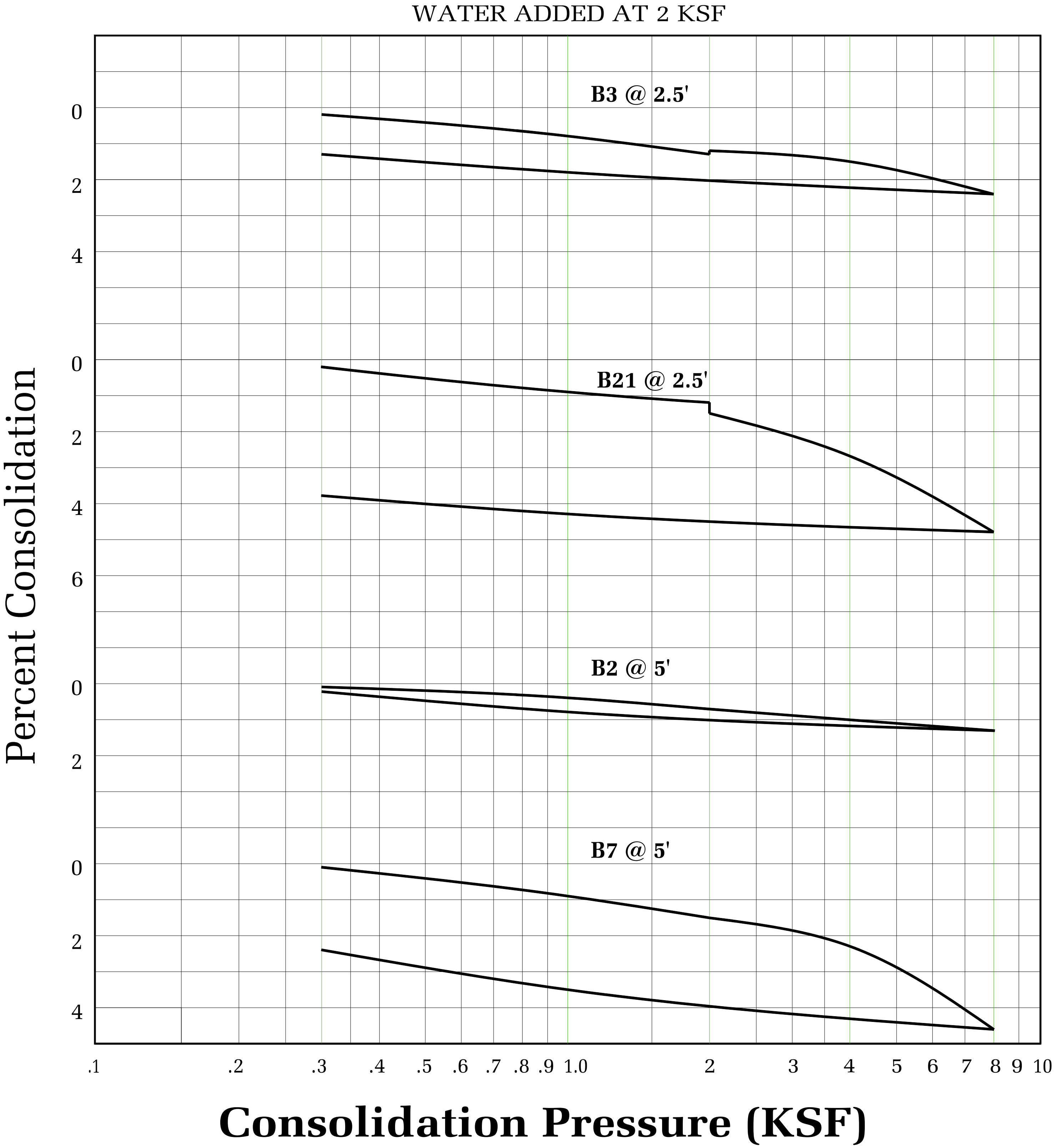
● Direct Shear, Saturated

SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

MGA NORTH, LLC

FILE NO. 20572 | PLATE: B-2



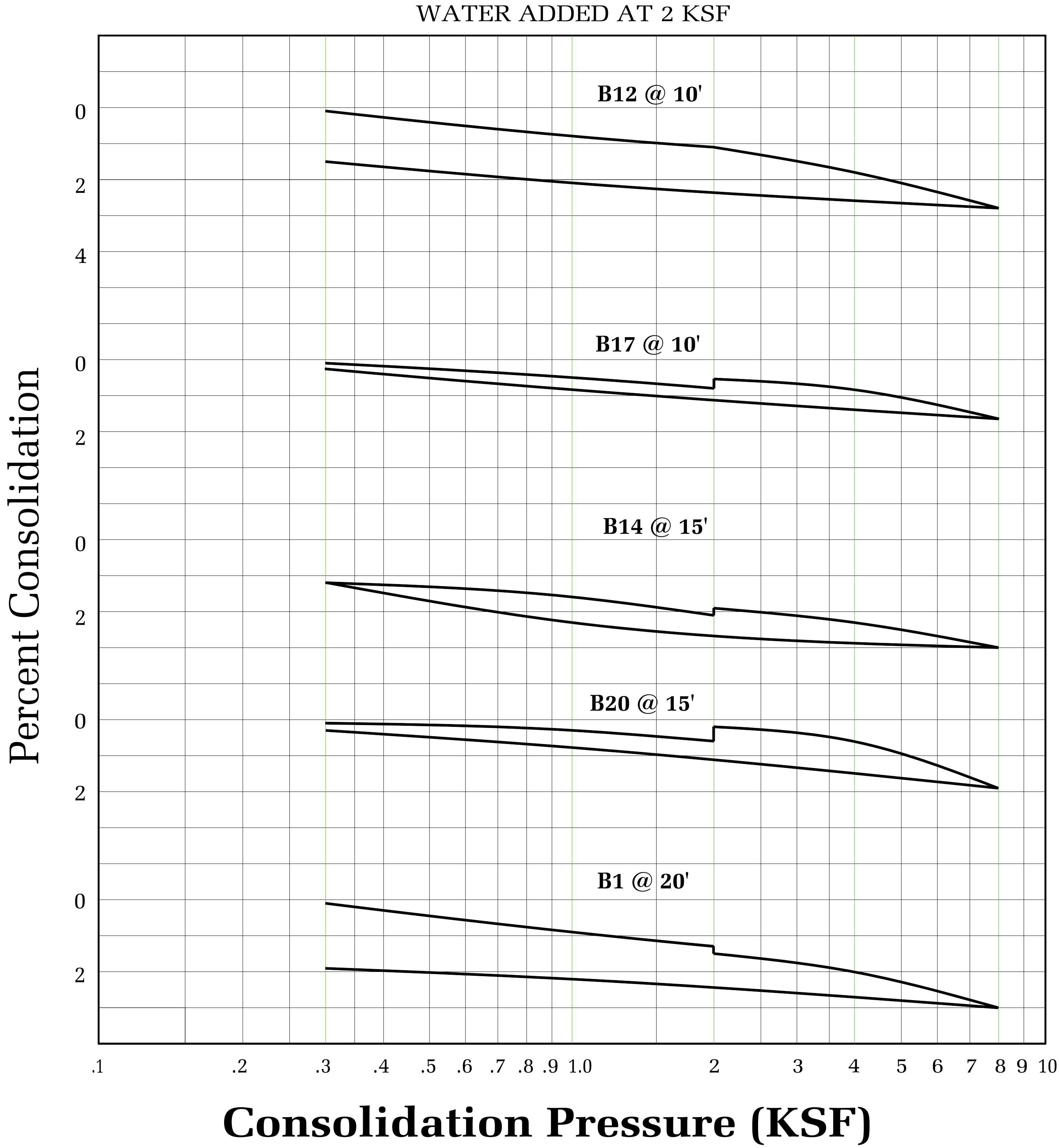
CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

MGA NORTH, LLC

FILE NO. 20572

PLATE: C-1

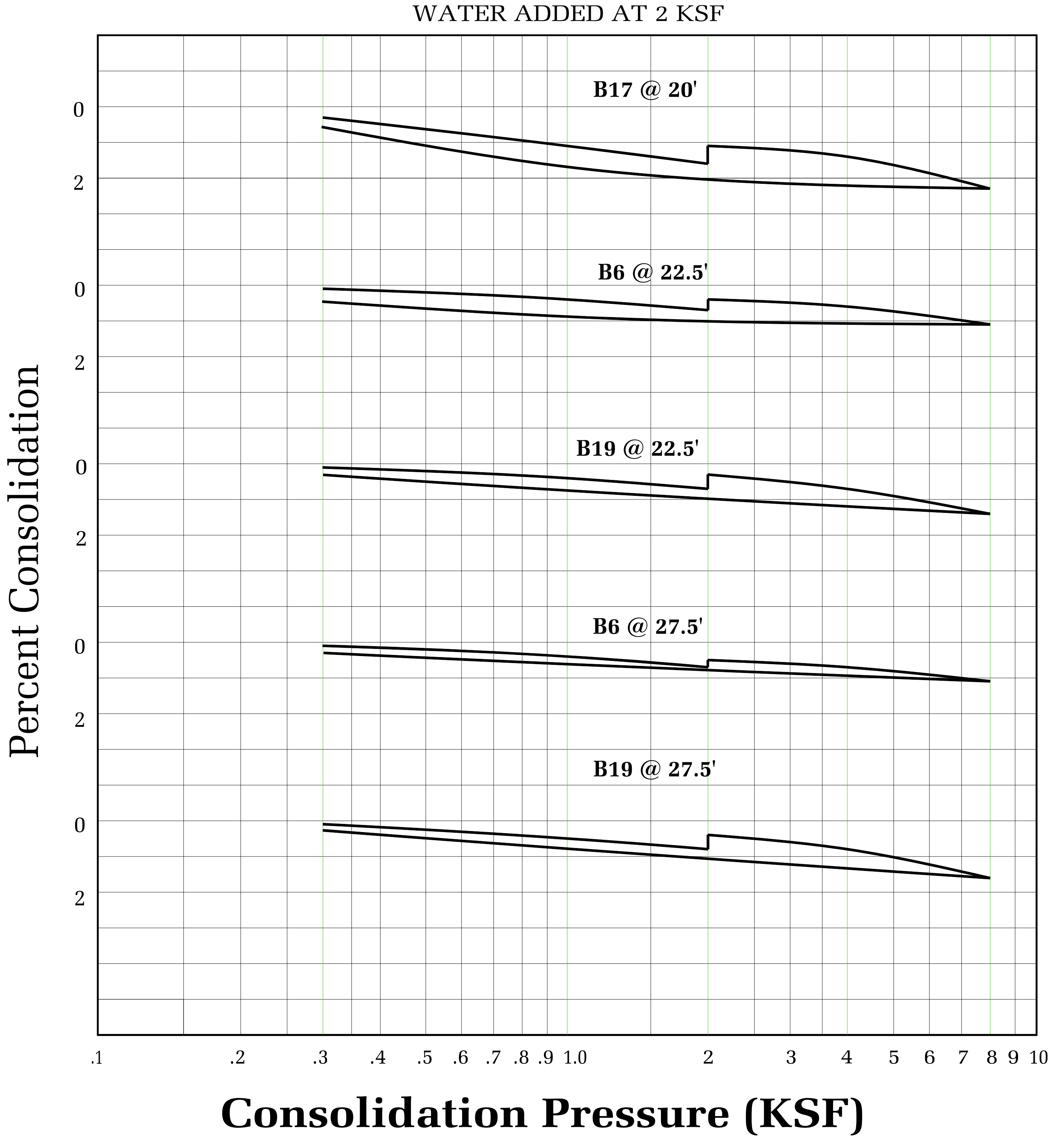


CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

MGA NORTH, LLC

FILE NO. 20572 | PLATE: C-2



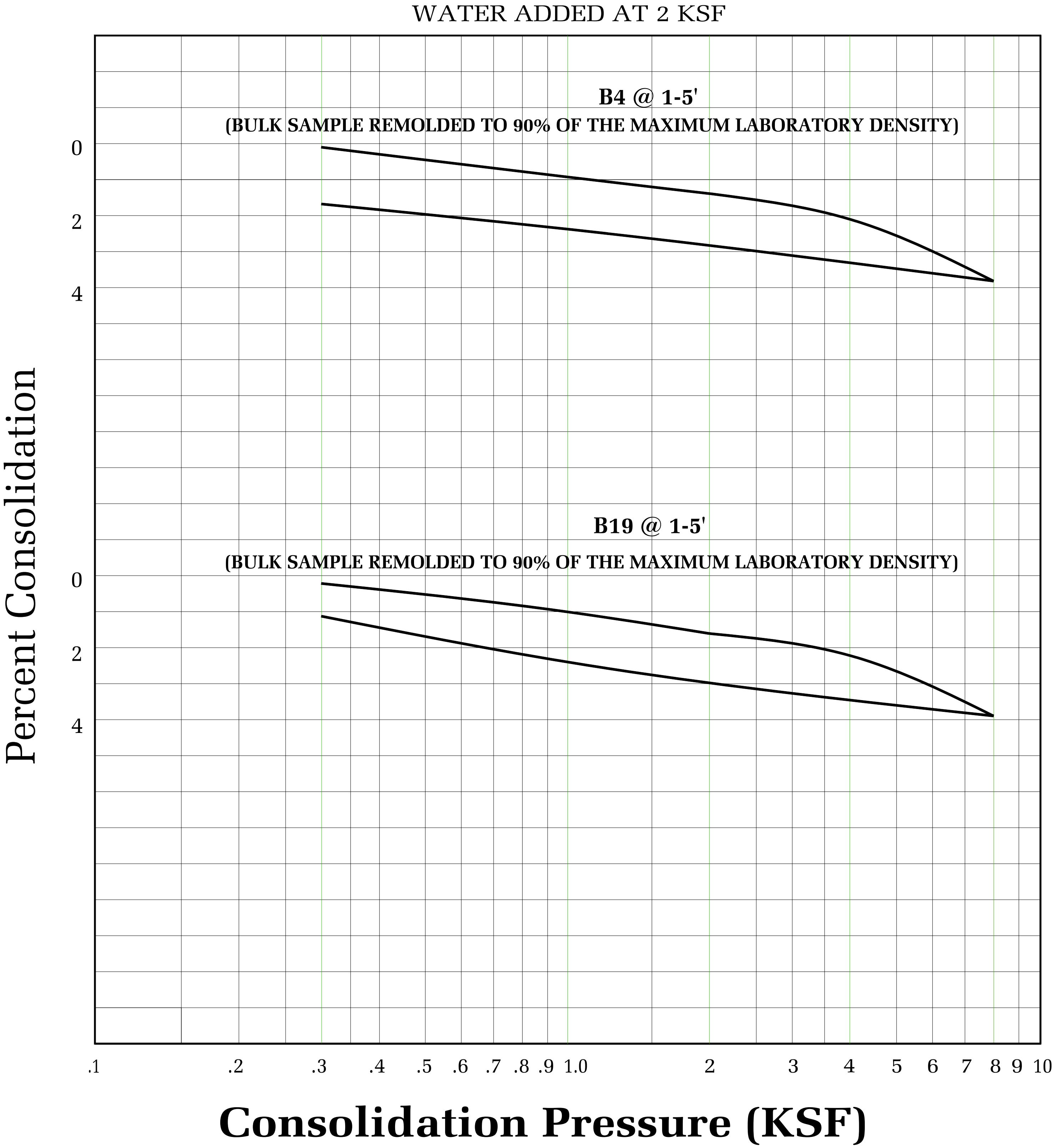
CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

MGA NORTH, LLC

FILE NO. 20572

PLATE: C-3



CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

MGA NORTH, LLC

FILE NO. 20572 | PLATE: C-4

ASTM D-1557

SAMPLE	B1 @ 1- 5'	B4 @ 1-5'	B12 @ 1-5'	B19 @ 1-5'
SOIL TYPE:	ML	SM/ML	SM + GRAVEL	SM/ML
MAXIMUM DENSITY pcf.	125.1	128.5	143.4	126.0
OPTIMUM MOISTURE %	11.4	9.1	5.5	10.4

ASTM D 4829-03

SAMPLE	B1 @ 1- 5'	B4 @ 1-5'	B12 @ 1-5'	B19 @ 1-5'
SOIL TYPE:	ML	SM/ML	SM + GRAVEL	SM/ML
EXPANSION INDEX UBC STANDARD 18-2	66	43	13	62
EXPANSION CHARACTER	MODERATE	LOW	VERY LOW	MODERATE

SULFATE CONTENT

SAMPLE	B1 @ 1- 5'	B4 @ 1-5'	B12 @ 1-5'	B19 @ 1-5'
SULFATE CONTENT: (percentage by weight)	< 0.10 %	< 0.10 %	< 0.10 %	< 0.10 %

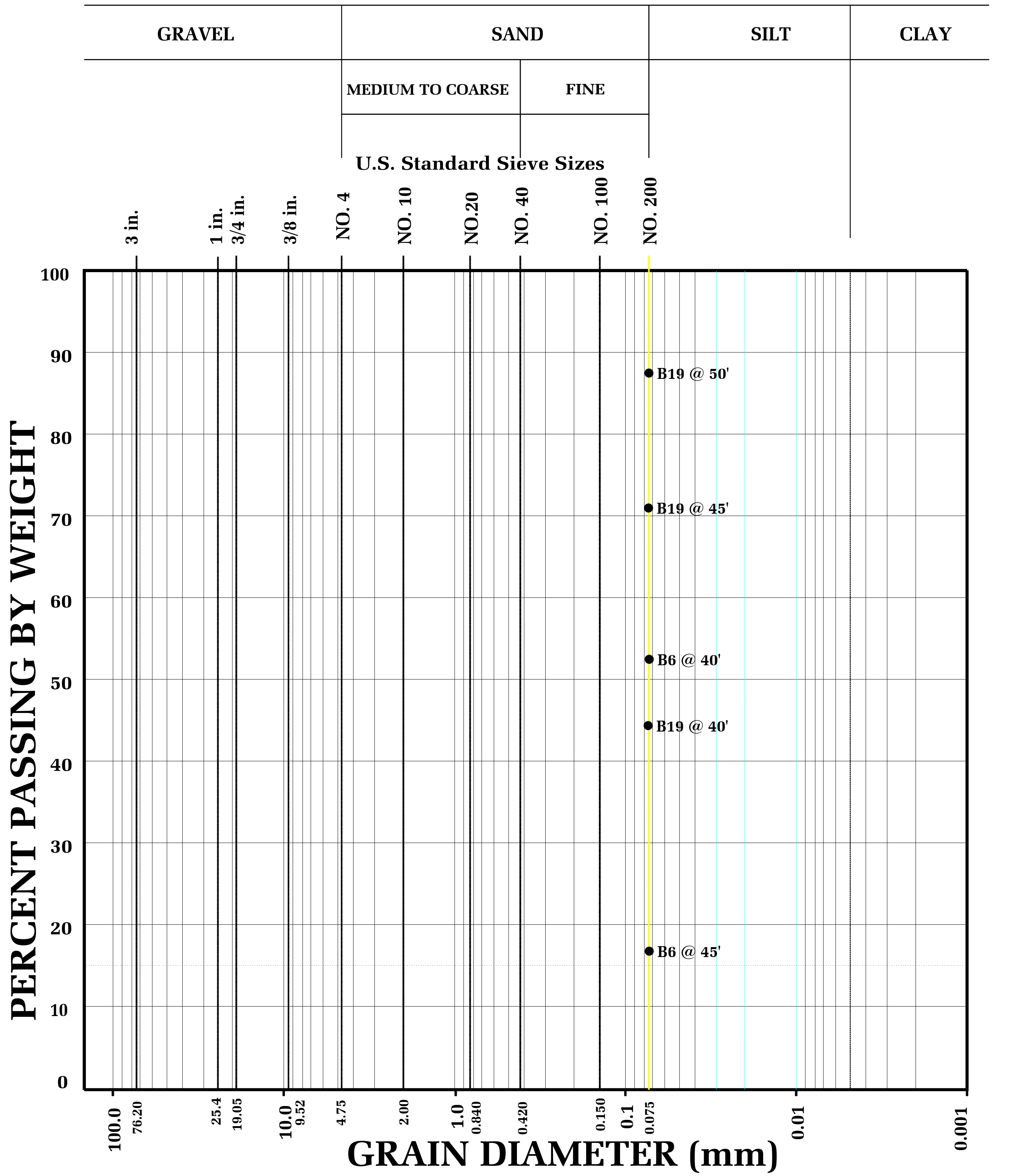
COMPACTION/EXPANSION/SULFATE DATA SHEET

Geotechnologies, Inc.
Consulting Geotechnical Engineers

MGA NORTH, LLC

FILE NO. 20572

PLATE: D



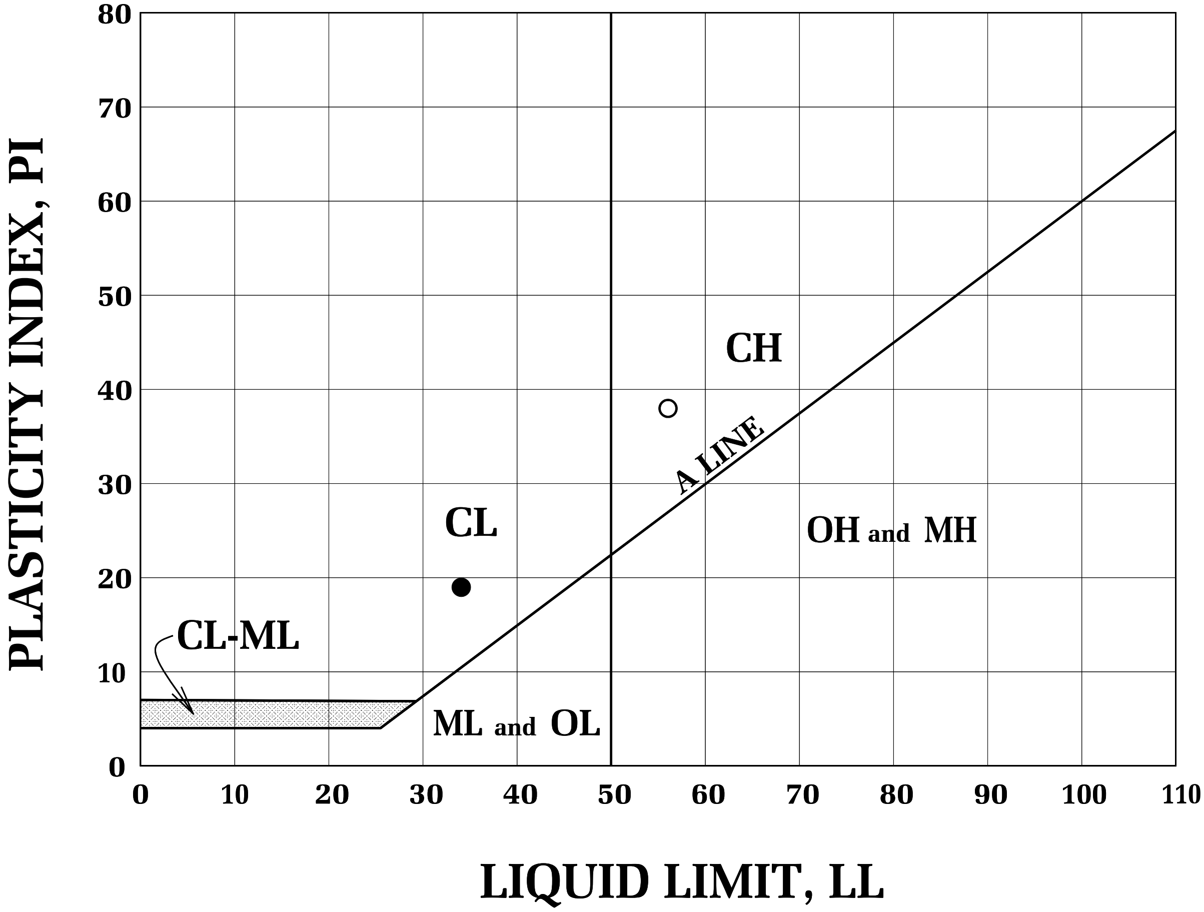
GRAIN SIZE DISTRIBUTION

Geotechnologies, Inc.
Consulting Geotechnical Engineers

MGA NORTH, LLC

FILE NO. 20572 | PLATE: E

ASTM D4318



BORING NUMBER	DEPTH (FEET)	TEST SYMBOL	LL	PL	PI	DESCRIPTION
B6	40	●	34	15	19	CL
B19	50	○	56	18	38	CH

ATTERBERG LIMITS DETERMINATION

Geotechnologies, Inc.
Consulting Geotechnical Engineers

MGA NORTH, LLC

FILE NO. 20572 | PLATE: F

Geotechnologies, Inc.

Project: MGA North, LLC
File No.: 20572
Description: Liquefaction Analysis
Boring Number: 6

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL

NCEER (1996) METHOD | By Thomas F. Blake (1994-1996) | LIQ2_30.WQ1

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.6
Peak Horiz. Acceleration (g):	0.48
Calculated Mag.Wtg.Factor:	0.724

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	51.0
Historic Highest Groundwater Level* (ft):	40.0
Unit Wt. Water (pcf):	62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

ENERGY & ROD CORRECTIONS:

Energy Correction (CE) for N60:	1.30
Rod Len.Corr.(CR)(0-no or 1-yes):	1.0
Bore Dia. Corr. (CB):	1.00
Sampler Corr. (CS):	1.20
Use Ksigma (0 or 1):	1.0

LIQUEFACTION CALCULATIONS:

Depth to Base (ft)	Total Unit Wt. (pcf)	Current Water Level (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected $(N_1)_{60}$	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	134.7	0	NA	1.0	0	0.0		2.000	0.0	~	0.998	0.226	~
2.0	134.7	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.993	0.224	~
3.0	134.7	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.989	0.223	~
4.0	134.7	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.984	0.222	~
5.0	134.7	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.979	0.221	~
6.0	134.7	0	26.0	5.0	0	0.0		1.868	56.8	~	0.975	0.220	~
7.0	134.7	0	26.0	5.0	0	0.0		1.868	56.8	~	0.970	0.219	~
8.0	136.4	0	26.0	5.0	0	0.0		1.868	56.8	~	0.966	0.218	~
9.0	136.4	0	26.0	5.0	0	0.0		1.868	56.8	~	0.961	0.217	~
10.0	136.4	0	26.0	5.0	0	0.0		1.868	56.8	~	0.957	0.216	~
11.0	136.4	0	14.0	10.0	0	0.0		1.284	21.0	~	0.952	0.215	~
12.0	136.4	0	14.0	10.0	0	0.0		1.284	21.0	~	0.947	0.214	~
13.0	124.8	0	14.0	10.0	0	0.0		1.284	21.0	~	0.943	0.213	~
14.0	124.8	0	14.0	10.0	0	0.0		1.284	21.0	~	0.938	0.212	~
15.0	124.8	0	14.0	10.0	0	0.0		1.284	21.0	~	0.934	0.211	~
16.0	124.5	0	8.0	15.0	0	0.0		1.045	10.5	~	0.929	0.210	~
17.0	124.5	0	8.0	15.0	0	0.0		1.045	10.5	~	0.925	0.209	~
18.0	124.5	0	8.0	15.0	0	0.0		1.045	10.5	~	0.920	0.208	~
19.0	124.5	0	8.0	15.0	0	0.0		1.045	10.5	~	0.915	0.207	~
20.0	124.5	0	8.0	15.0	0	0.0		1.045	10.5	~	0.911	0.206	~
21.0	124.5	0	11.0	20.0	0	0.0		0.909	14.0	~	0.906	0.205	~
22.0	124.5	0	11.0	20.0	0	0.0		0.909	14.0	~	0.902	0.204	~
23.0	128.6	0	11.0	20.0	0	0.0		0.909	14.0	~	0.897	0.203	~
24.0	128.6	0	11.0	25.0	0	0.0		0.814	13.3	~	0.893	0.202	~
25.0	128.6	0	11.0	25.0	0	0.0		0.814	13.3	~	0.888	0.201	~
26.0	128.6	0	11.0	25.0	0	0.0		0.814	13.3	~	0.883	0.200	~
27.0	128.6	0	11.0	25.0	0	0.0		0.814	13.3	~	0.879	0.199	~
28.0	129.4	0	11.0	25.0	0	0.0		0.814	13.3	~	0.874	0.198	~
29.0	129.4	0	11.0	25.0	0	0.0		0.814	13.3	~	0.870	0.197	~
30.0	129.4	0	11.0	25.0	0	0.0		0.814	13.3	~	0.865	0.196	~
31.0	129.4	0	14.0	30.0	0	0.0		0.743	16.2	~	0.861	0.195	~
32.0	129.4	0	14.0	30.0	0	0.0		0.743	16.2	~	0.856	0.193	~
33.0	135.1	0	11.0	35.0	0	0.0		0.686	11.8	~	0.851	0.192	~
34.0	135.1	0	11.0	35.0	0	0.0		0.686	11.8	~	0.847	0.191	~
35.0	135.1	0	11.0	35.0	0	0.0		0.686	11.8	~	0.842	0.190	~
36.0	135.1	0	11.0	35.0	0	0.0		0.686	11.8	~	0.838	0.189	~
37.0	135.1	0	11.0	35.0	0	0.0		0.686	11.8	~	0.833	0.188	~
38.0	129.6	0	11.0	35.0	0	0.0		0.686	11.8	~	0.829	0.187	~
39.0	129.6	0	11.0	35.0	0	0.0		0.686	11.8	~	0.824	0.186	~
40.0	129.6	0	14.0	35.0	0	0.0		0.686	15.0	~	0.819	0.185	~
41.0	129.6	0	14.0	40.0	0	56.5		0.640	21.0	~	0.815	0.184	~
42.0	129.6	0	14.0	40.0	0	56.5		0.640	21.0	~	0.810	0.183	~
43.0	111.9	0	31.0	45.0	1	16.7	71	0.606	32.0	Infin.	0.806	0.182	Non-Liq.
44.0	111.9	0	31.0	45.0	1	16.7	71	0.606	32.0	Infin.	0.801	0.181	Non-Liq.
45.0	111.9	0	31.0	45.0	1	16.7	71	0.606	32.0	Infin.	0.797	0.180	Non-Liq.
46.0	111.9	0	31.0	45.0	1	16.7	71	0.606	32.0	Infin.	0.792	0.179	Non-Liq.
47.0	111.9	0	31.0	45.0	1	16.7	71	0.606	32.0	Infin.	0.787	0.178	Non-Liq.
48.0	114.5	0	65.0	50.0	1	16.7	99	0.600	63.6	Infin.	0.783	0.177	Non-Liq.
49.0	114.5	0	65.0	50.0	1	16.7	99	0.600	63.6	Infin.	0.778	0.176	Non-Liq.
50.0	114.5	0	65.0	50.0	1	16.7	99	0.600	63.6	Infin.	0.774	0.175	Non-Liq.

Geotechnologies, Inc.

Project: MGA North, LLC
File No.: 20572
Description: Liquefaction Analysis
Boring Number: 19

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL

NCEER (1996) METHOD

By Thomas F. Blake (1994-1996)

LIQ2_30.WQ1

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.6
Peak Horiz. Acceleration (g):	0.48
Calculated Mag.Wtg.Factor:	0.724

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	51.0
Historic Highest Groundwater Level* (ft):	40.0
Unit Wt. Water (pcf):	62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

ENERGY & ROD CORRECTIONS:

Energy Correction (CE) for N60:	1.30
Rod Len.Corr.(CR)(0-no or 1-yes):	1.0
Bore Dia. Corr. (CB):	1.00
Sampler Corr. (CS):	1.20
Use Ksigma (0 or 1):	1.0

LIQUEFACTION CALCULATIONS:

Depth to Base (ft)	Total Unit Wt. (pcf)	Current Water Level (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected $(N_1)_{60}$	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	127.8	0	NA	1.0	0	0.0		2.000	0.0	~	0.998	0.226	~
2.0	127.8	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.993	0.224	~
3.0	127.8	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.989	0.223	~
4.0	127.8	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.984	0.222	~
5.0	127.8	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.979	0.221	~
6.0	127.8	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.975	0.220	~
7.0	127.8	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.970	0.219	~
8.0	125.6	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.966	0.218	~
9.0	125.6	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.961	0.217	~
10.0	125.6	0	NA	1.0	0	0.0		#######	#VALUE!	~	0.957	0.216	~
11.0	125.6	0	17.0	10.0	0	0.0		1.323	26.3	~	0.952	0.215	~
12.0	125.6	0	17.0	10.0	0	0.0		1.323	26.3	~	0.947	0.214	~
13.0	115.8	0	27.0	15.0	0	0.0		1.081	36.7	~	0.943	0.213	~
14.0	115.8	0	27.0	15.0	0	0.0		1.081	36.7	~	0.938	0.212	~
15.0	115.8	0	27.0	15.0	0	0.0		1.081	36.7	~	0.934	0.211	~
16.0	115.8	0	27.0	15.0	0	0.0		1.081	36.7	~	0.929	0.210	~
17.0	115.8	0	27.0	15.0	0	0.0		1.081	36.7	~	0.925	0.209	~
18.0	105.0	0	27.0	15.0	0	0.0		1.081	36.7	~	0.920	0.208	~
19.0	105.0	0	27.0	15.0	0	0.0		1.081	36.7	~	0.915	0.207	~
20.0	105.0	0	27.0	15.0	0	0.0		1.081	36.7	~	0.911	0.206	~
21.0	118.5	0	14.0	20.0	0	0.0		0.946	18.5	~	0.906	0.205	~
22.0	118.5	0	14.0	20.0	0	0.0		0.946	18.5	~	0.902	0.204	~
23.0	118.5	0	14.0	20.0	0	0.0		0.946	18.5	~	0.897	0.203	~
24.0	118.5	0	14.0	20.0	0	0.0		0.946	18.5	~	0.893	0.202	~
25.0	118.5	0	14.0	20.0	0	0.0		0.946	18.5	~	0.888	0.201	~
26.0	118.5	0	13.0	25.0	0	0.0		0.847	16.4	~	0.883	0.200	~
27.0	118.5	0	13.0	25.0	0	0.0		0.847	16.4	~	0.879	0.199	~
28.0	115.1	0	13.0	25.0	0	0.0		0.847	16.4	~	0.874	0.198	~
29.0	115.1	0	13.0	25.0	0	0.0		0.847	16.4	~	0.870	0.197	~
30.0	115.1	0	13.0	25.0	0	0.0		0.847	16.4	~	0.865	0.196	~
31.0	115.1	0	19.0	30.0	0	0.0		0.774	22.9	~	0.861	0.195	~
32.0	115.1	0	19.0	30.0	0	0.0		0.774	22.9	~	0.856	0.193	~
33.0	112.1	0	19.0	30.0	0	0.0		0.774	22.9	~	0.851	0.192	~
34.0	112.1	0	19.0	30.0	0	0.0		0.774	22.9	~	0.847	0.191	~
35.0	112.1	0	19.0	30.0	0	0.0		0.774	22.9	~	0.842	0.190	~
36.0	112.1	0	14.0	35.0	0	0.0		0.718	15.7	~	0.838	0.189	~
37.0	112.1	0	14.0	35.0	0	0.0		0.718	15.7	~	0.833	0.188	~
38.0	114.5	0	14.0	35.0	0	0.0		0.718	15.7	~	0.829	0.187	~
39.0	114.5	0	14.0	35.0	0	0.0		0.718	15.7	~	0.824	0.186	~
40.0	114.5	0	14.0	35.0	0	0.0		0.718	15.7	~	0.819	0.185	~
41.0	114.5	0	20.0	40.0	1	44.4	61	0.673	28.0	0.304	0.815	0.184	1.65
42.0	114.5	0	20.0	40.0	1	44.4	61	0.673	28.0	0.304	0.810	0.183	1.66
43.0	133.2	0	20.0	40.0	1	44.4	61	0.673	28.0	0.304	0.806	0.182	1.67
44.0	133.2	0	20.0	40.0	1	44.4	61	0.673	28.0	0.304	0.801	0.181	1.68
45.0	133.2	0	20.0	40.0	1	44.4	61	0.673	28.0	0.304	0.797	0.180	1.69
46.0	133.2	0	17.0	45.0	0	71.0		0.633	23.8	~	0.792	0.179	~
47.0	133.2	0	17.0	45.0	0	71.0		0.633	23.8	~	0.787	0.178	~
48.0	134.3	0	21.0	50.0	0	87.5		0.600	26.7	~	0.783	0.177	~
49.0	134.3	0	21.0	50.0	0	87.5		0.600	26.7	~	0.778	0.176	~
50.0	134.3	0	21.0	50.0	0	87.5		0.600	26.7	~	0.774	0.175	~

Geotechnologies, Inc.

FILE NO.: 20572
PROJECT: MGA North, LLC

EVALUATION OF EARTHQUAKE-INDUCED SETTLEMENTS IN DRY SANDY SOILS

INPUT:

Boring No.: 6
Depth to Groundwater: > 50 feet

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.6
Peak Horiz. Acceleration (g):	0.48

Depth of Base of Strata (ft)	Thickness of Layer (ft)	USCS Classification	Depth of Mid-point of Layer (ft)	Soil Unit Weight (pcf)	Overburden Pressure at Mid-point (tsf)	Mean Effective Pressure at Mid-point (tsf)	Average Cyclic Shear Stress [Tav]	Field SPT [N]	Correction Factor [Cer]	Relative Density [Dr] (%)	Correction Factor [Cn]	Corrected [N1]60	Maximum Shear Mod. [Gmax] (tsf)	[geff]*[Geff] [Gmax]	From Tbl. 4-4 [geff]	[geff]*100%	From Tbl. 4-5 Volumetric Strain [E15] (%)	Number of Strain Cycles [Nc]	Corrected Vol. Strains [Ec]	Settlement [S] (inches)
5.0	5.0	SM/ML	2.5	134.7	0.17	0.11	0.053	N/A					COMPACTED FILL PAD							
10.0	5.0	SM/ML	7.5	136.4	0.51	0.34	0.157	26	1.3	100.0	1.30	43.9	919.551	1.49E-04	3.20E-04	3.20E-02	7.00E-03	7.9523	0.0053	0.01
15.0	5.0	SM/ML	12.5	124.8	0.83	0.56	0.256	14	1.3	68.0	1.13	20.5	913.982	2.25E-04	5.80E-04	5.80E-02	5.50E-02	7.9523	0.0413	0.05
20.0	5.0	ML	17.5	124.5	1.15	0.77	0.347	8	1.3	48.0	0.95	9.9	840.242	3.08E-04	9.00E-04	9.00E-02	1.50E-01	7.9523	0.1127	0.14
30.0	10.0	SM/ML	25.0	128.6	1.62	1.09	0.476	11	1.3	50.0	0.79	11.3	1045.743	3.10E-04	7.20E-04	7.20E-02	1.40E-01	7.9523	0.1052	0.25
32.5	2.5	SM/ML	31.3	129.4	2.02	1.36	0.576	14	1.3	57.0	0.70	12.7	1216.001	3.02E-04	6.30E-04	6.30E-02	1.20E-01	7.9523	0.0902	0.05
40.0	7.5	SM/ML	36.3	135.1	2.36	1.58	0.651	11	1.3	46.0	0.64	9.2	1175.493	3.37E-04	7.30E-04	7.30E-02	2.00E-01	7.9523	0.1503	0.27
42.5	2.5	CH	41.3	129.6	2.69	1.80	0.718	14	1.3	51.0	0.58	10.6	1317.224	3.20E-04	6.30E-04	6.30E-02	1.50E-01	7.9523	0.1127	0.07
45.0	2.5	SM/ML	43.8	111.9	2.84	1.91	0.745	31	1.3	74.0	0.78	31.4	1947.452	2.21E-04	3.35E-04	3.35E-02	1.80E-02	7.9523	0.0135	0.01
50.0	5.0	SP/SW	47.5	114.5	3.06	2.05	0.778	65	1.3	98.0	0.60	50.7	2367.856	1.85E-04	2.35E-04	2.35E-02	3.80E-03	7.9523	0.0029	0.00

Total Earthquake-Induced Settlements in Dry Sandy Soils (inches) = 0.85

Geotechnologies, Inc.

FILE NO.: 20572
PROJECT: MGA North, LLC

EVALUATION OF EARTHQUAKE-INDUCED SETTLEMENTS IN DRY SANDY SOILS

INPUT:

Boring No.: 19
Depth to Groundwater: > 50 feet

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.6
Peak Horiz. Acceleration (g):	0.48

Depth of Base of Strata (ft)	Thickness of Layer (ft)	USCS Classification	Depth of Mid-point of Layer (ft)	Soil Unit Weight (pcf)	Overburden Pressure at Mid-point (tsf)	Mean Effective Pressure at Mid-point (tsf)	Average Cyclic Shear Stress [Tav]	Field SPT [N]	Correction Factor [Cer]	Relative Density [Dr] (%)	Correction Factor [Cn]	Corrected [N1]60	Maximum Shear Mod. [Gmax] (tsf)	[geff]*[Geff] [Gmax]	From Tbl. 4-4 [geff]	[geff]*100%	From Tbl. 4-5 Volumetric Strain [E15) (%)	Number of Strain Cycles [Nc]	Corrected Vol. Strains [Ec]	Settlement [S] (inches)
6.0	6.0	SM/ML	3.0	127.8	0.19	0.13	0.060	N/A					COMPACTED FILL PAD							
12.5	6.5	SM/ML/SP	9.3	125.6	0.59	0.39	0.182	17	1.3	80.0	1.28	28.3	854.508	1.80E-04	4.20E-04	4.20E-02	3.00E-02	7.9523	0.0225	0.04
20.0	7.5	SM/ML/SP	16.3	115.8	1.01	0.68	0.307	27	1.3	90.0	1.00	35.1	1203.189	1.94E-04	3.80E-04	3.80E-02	1.60E-02	7.9523	0.0120	0.02
30.0	10.0	ML	25.0	118.5	1.52	1.02	0.447	14	1.3	60.0	0.82	14.9	1111.377	2.73E-04	5.70E-04	5.70E-02	9.00E-02	7.9523	0.0676	0.16
35.0	5.0	SM/ML	32.5	112.1	1.96	1.31	0.553	19	1.3	66.0	0.76	18.8	1360.841	2.56E-04	5.00E-04	5.00E-02	5.60E-02	7.9523	0.0421	0.05
40.0	5.0	SM/SP	37.5	114.5	2.24	1.50	0.613	14	1.3	55.0	0.64	11.6	1241.804	2.98E-04	5.60E-04	5.60E-02	1.20E-01	7.9523	0.0902	0.11
45.0	5.0	ML	42.5	114.5	2.53	1.69	0.668	20	1.3	62.0	0.66	17.2	1500.504	2.59E-04	4.60E-04	4.60E-02	6.20E-02	7.9523	0.0466	0.06
50.0	5.0	CH	47.5	134.3	2.84	1.90	0.723	21	1.3	62.0	0.62	16.9	1582.865	2.57E-04	4.10E-04	4.10E-02	6.00E-02	7.9523	0.0451	0.05

Total Earthquake-Induced Settlements in Dry Sandy Soils (inches) = 0.49

Geotechnologies, Inc.

Project: MGA North, LLC
File No.: 20572
Description: Foundation Pile Design

6 feet of Non Bearing Soils

Drilled Friction Pile Capacity Calculation

Input Data:

Unit Weight of Overlying Soil Layer	120	pcf
Thickness of Overlying Soil Layer	6	feet
Unit Weight of Bearing Strata	120	pcf
Friction Angle of Bearing Strata	28	degrees
Friction Angle between Pile and Soil	21	degrees
Cohesion of Bearing Strata	0	psf
Adhesion	0	psf
Minimum Embedment into Bearing Strata	10	feet
Unit Weight of Water	62.4	pcf
Depth to Groundwater from Pile Cap	60	feet

Pile Design:
Drilled <<Driven/Drilled
Circular <<Circular/Square Pile

Pile Dimension:
18 inch diameter pile
24 inch diameter pile

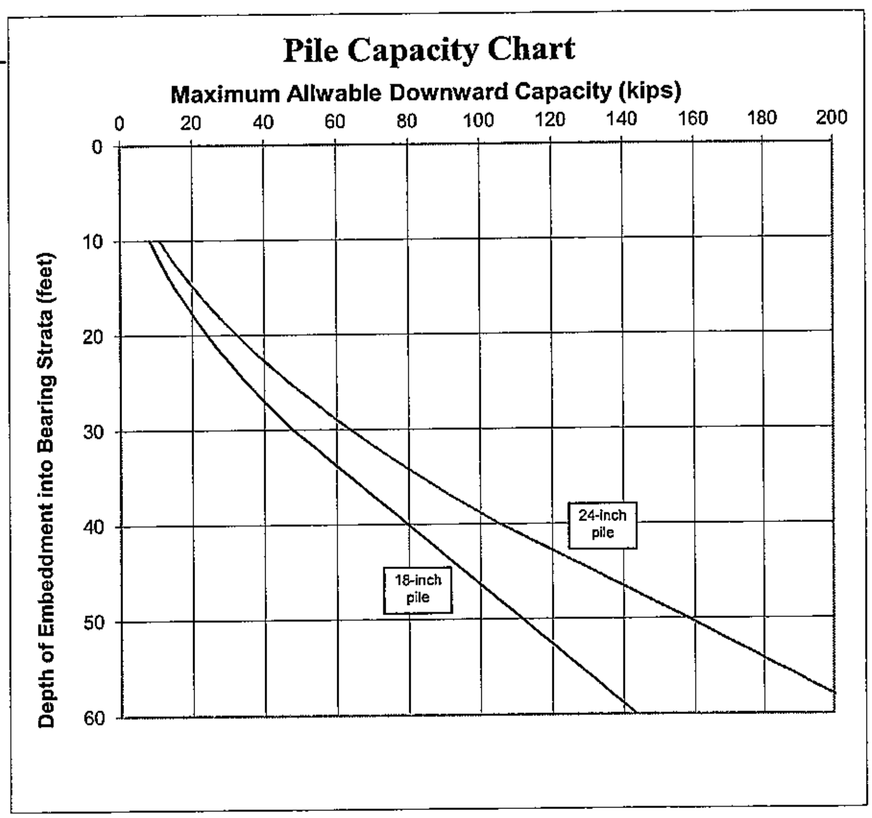
Critical Depth Limit (Dc):
20 B

Lateral Earth Pressure Coefficient: 0.70
Applied Factor of Safety: 2
Factored Skin Friction

Note: 1. Minimum pile embeddment depth of 10 feet
2. Uplift capacity may be designed using 50% of the downward capacity
3. Pile should be spaced a minimum of 3 diameters on center
4. See text of report for pile details and installation recommendations

Pile Capacity:

Total Depth of Pile (feet)	Depth of Embeddment into Bearing Strata (feet)	Maximum Allowable Downward Pile Capacity: Capacity of 18 diameter pile (kips)	Capacity of 24 diameter pile (kips)
16	10	8.4	11.1
17	11	9.6	12.8
18	12	10.9	14.6
19	13	12.3	16.5
20	14	13.8	18.4
21	15	15.4	20.5
22	16	17.0	22.7
23	17	18.7	25.0
24	18	20.5	27.4
25	19	22.4	29.8
26	20	24.3	32.4
27	21	26.3	35.1
28	22	28.4	37.9
29	23	30.6	40.8
30	24	32.8	43.8
31	25	35.1	46.9
32	26	37.5	50.0
33	27	40.0	53.3
34	28	42.5	56.7
35	29	45.2	60.2
36	30	47.9	63.8
37	31	51.1	67.5
38	32	54.2	71.3
39	33	57.4	75.2
40	34	60.6	79.2
41	35	63.8	83.3
42	36	67.0	87.5
43	37	70.2	91.8
44	38	73.4	96.2
45	39	76.6	100.7
46	40	79.8	105.4
47	41	83.0	110.6
48	42	86.2	115.9
49	43	89.3	121.2
50	44	92.5	126.4
51	45	95.7	131.7
52	46	98.9	137.0
53	47	102.1	142.2
54	48	105.3	147.5
55	49	108.5	152.8
56	50	111.7	158.0
57	51	114.9	163.3
58	52	118.1	168.6
59	53	121.3	173.8
60	54	124.4	179.1



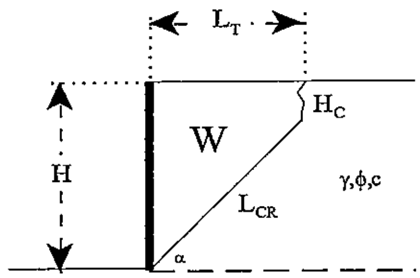
Geotechnologies, Inc.

Project: MGA North, LLC
File No.: 20572
Description: Cantilever Retaining Walls (Up to 6 feet)

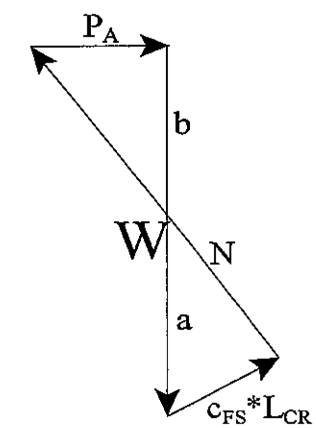
Retaining Wall Design with Level Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	6.00 feet
Unit Weight of Retained Soils	(γ)	125.0 pcf
Friction Angle of Retained Soils	(ϕ)	28.0 degrees
Cohesion of Retained Soils	(c)	150.0 psf
Factor of Safety	(FS)	1.50
Factored Parameters:	(ϕ_{FS})	19.5 degrees
	(c_{FS})	100.0 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	2.8	17	2092.0	5.0	1335.5	756.5	282.6
41	2.7	16	2053.2	5.0	1283.5	769.6	302.9
42	2.7	16	2010.2	5.0	1232.8	777.4	321.7
43	2.6	16	1964.1	5.0	1183.6	780.5	339.1
44	2.5	15	1915.9	5.0	1136.3	779.5	355.0
45	2.5	15	1866.0	5.0	1091.0	775.0	369.4
46	2.4	15	1815.2	5.0	1047.8	767.4	382.3
47	2.4	14	1763.6	4.9	1006.5	757.1	393.8
48	2.4	14	1711.7	4.9	967.3	744.4	403.9
49	2.3	13	1659.6	4.9	929.9	729.6	412.5
50	2.3	13	1607.5	4.8	894.4	713.1	419.7
51	2.3	12	1555.6	4.8	860.6	695.0	425.6
52	2.3	12	1503.9	4.7	828.4	675.5	430.1
53	2.3	12	1452.6	4.7	797.7	654.9	433.1
54	2.3	11	1401.6	4.6	768.4	633.2	434.9
55	2.3	11	1351.0	4.6	740.4	610.6	435.2
56	2.3	10	1300.8	4.5	713.6	587.2	434.2
57	2.3	10	1251.1	4.4	687.9	563.1	431.8
58	2.3	10	1201.7	4.4	663.2	538.5	428.1
59	2.3	9	1152.8	4.3	639.4	513.4	423.0
60	2.3	9	1104.3	4.2	616.4	487.9	416.4
61	2.3	8	1056.2	4.2	594.1	462.0	408.5
62	2.4	8	1008.4	4.1	572.5	435.9	399.2
63	2.4	8	960.9	4.0	551.3	409.6	388.4
64	2.5	7	913.7	3.9	530.6	383.1	376.3
65	2.5	7	866.7	3.9	510.2	356.6	362.6



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+\phi_{FS})/\sin(\alpha-\phi_{FS})$

$b = W-a$

$P_A = b*\tan(\alpha-\phi_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

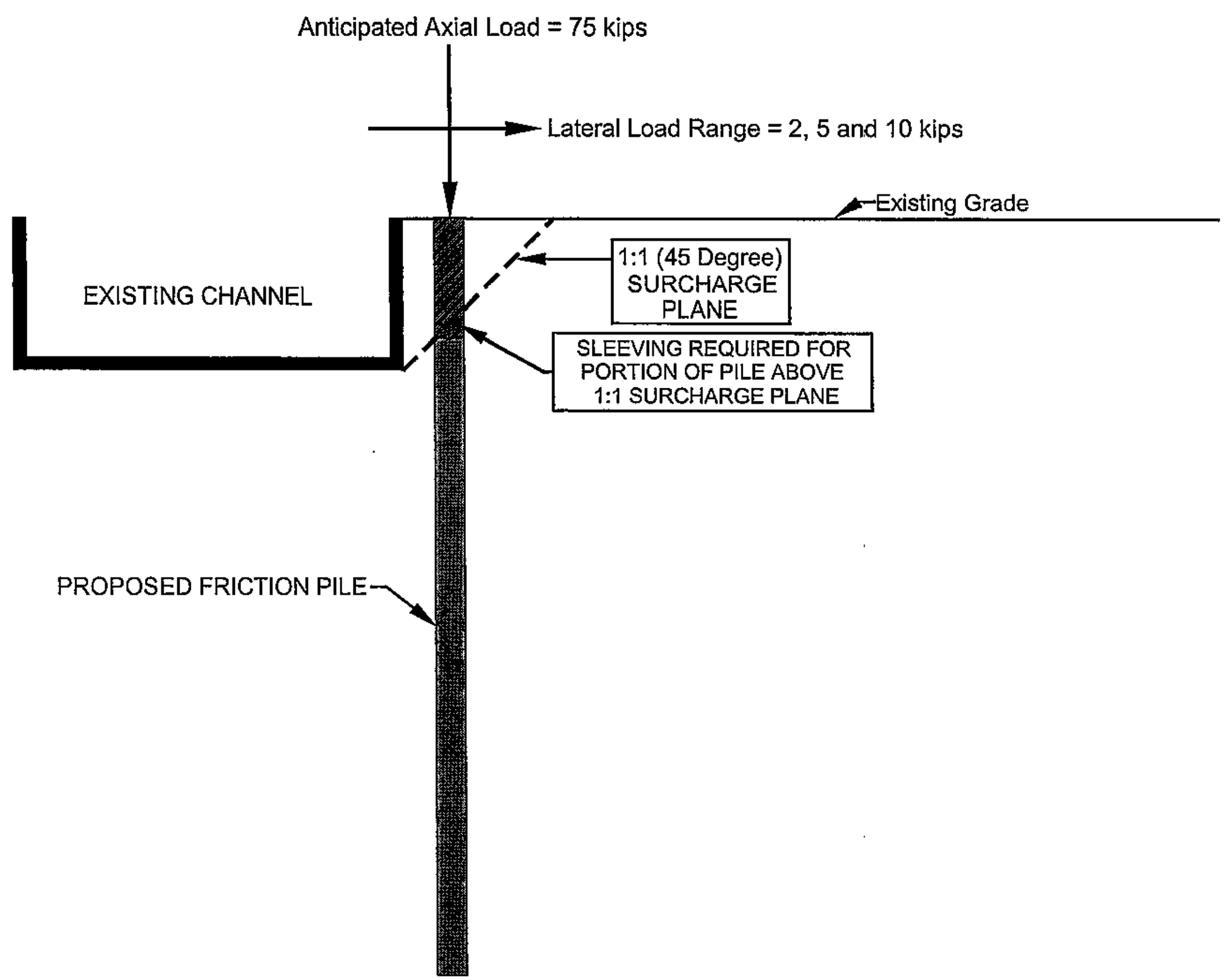
$P_{A,\max}$ 435.2 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

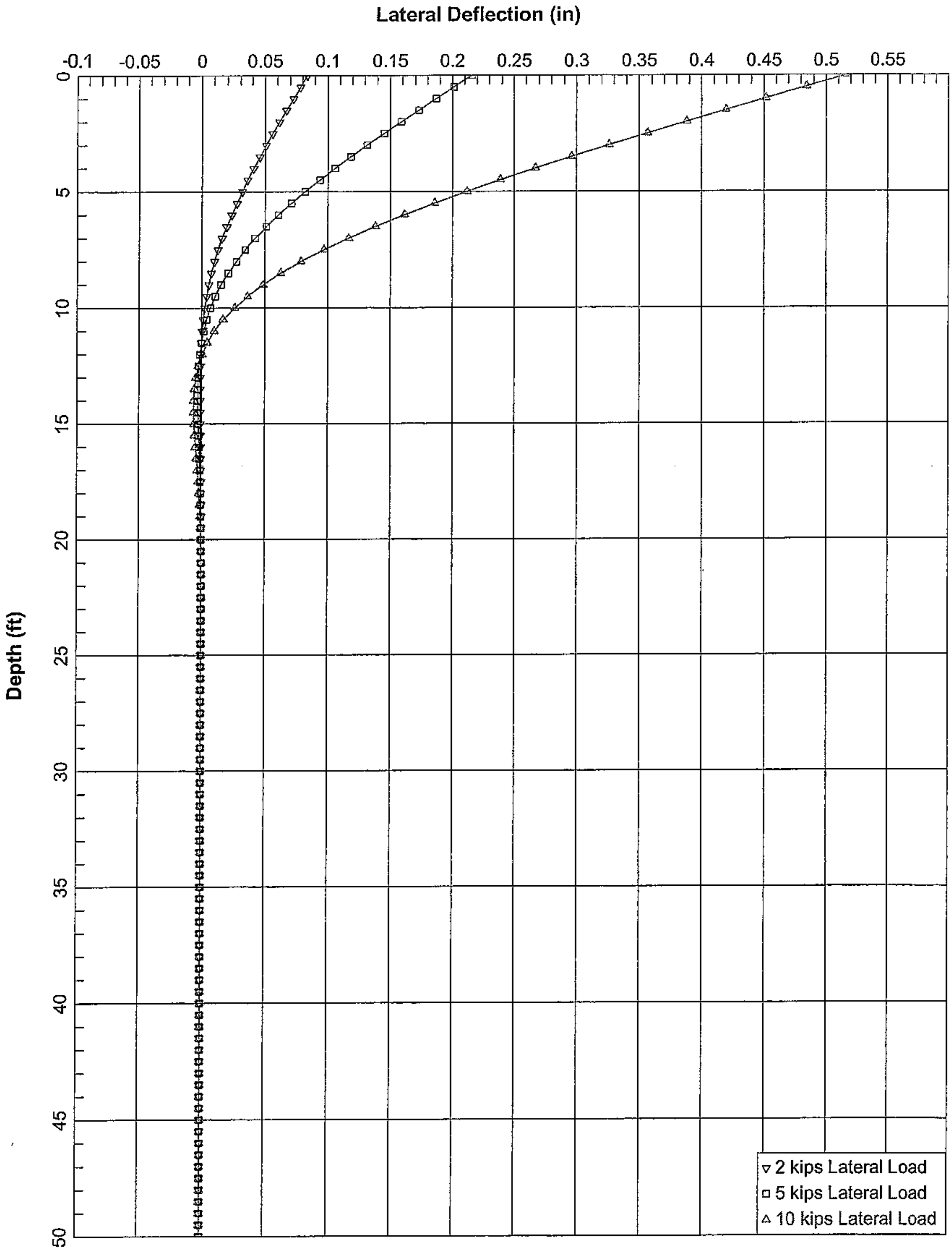
$EFP = 2*P_A/H^2$

EFP 24.2 pcf

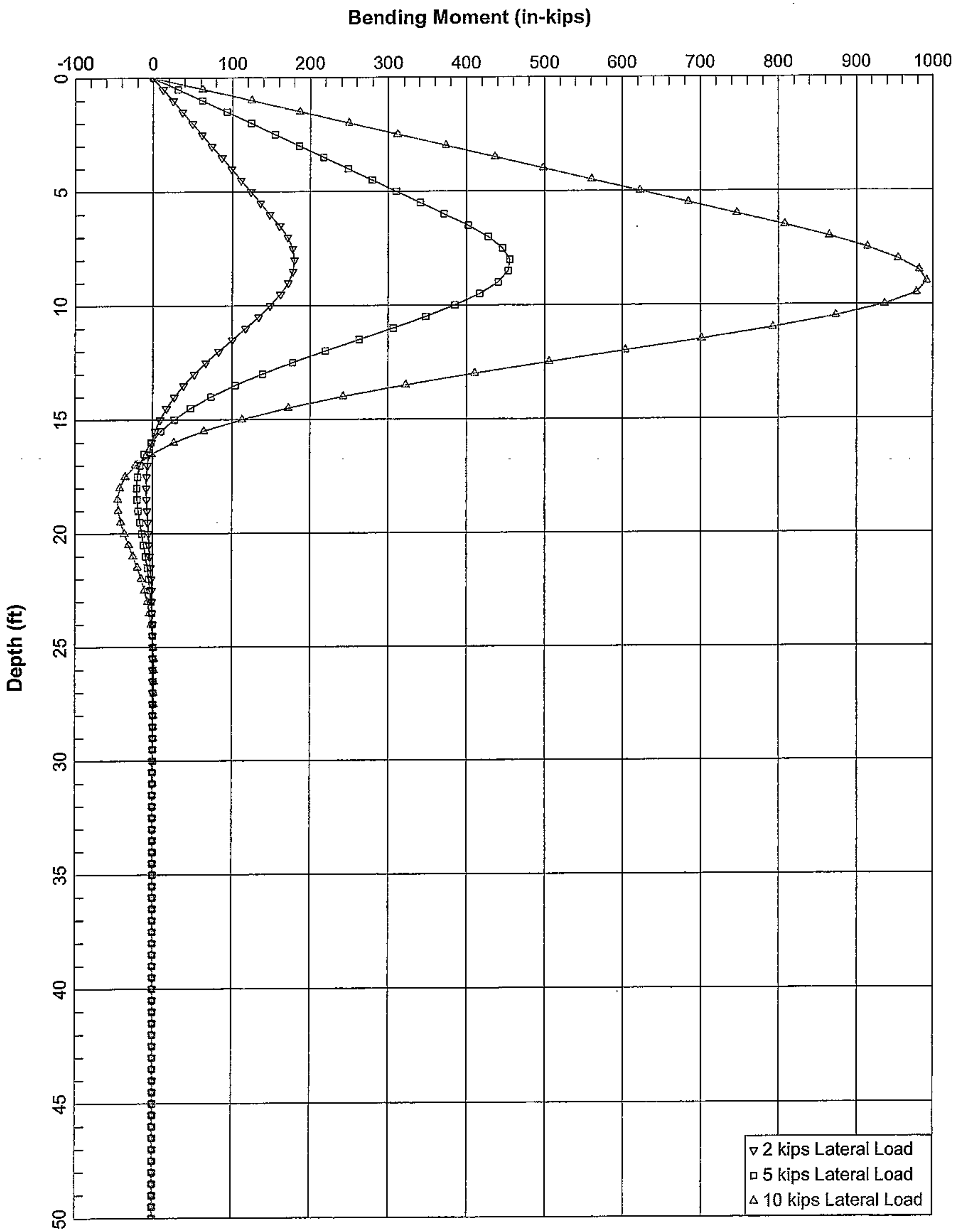
Design Wall for an Equivalent Fluid Pressure: 30 pcf



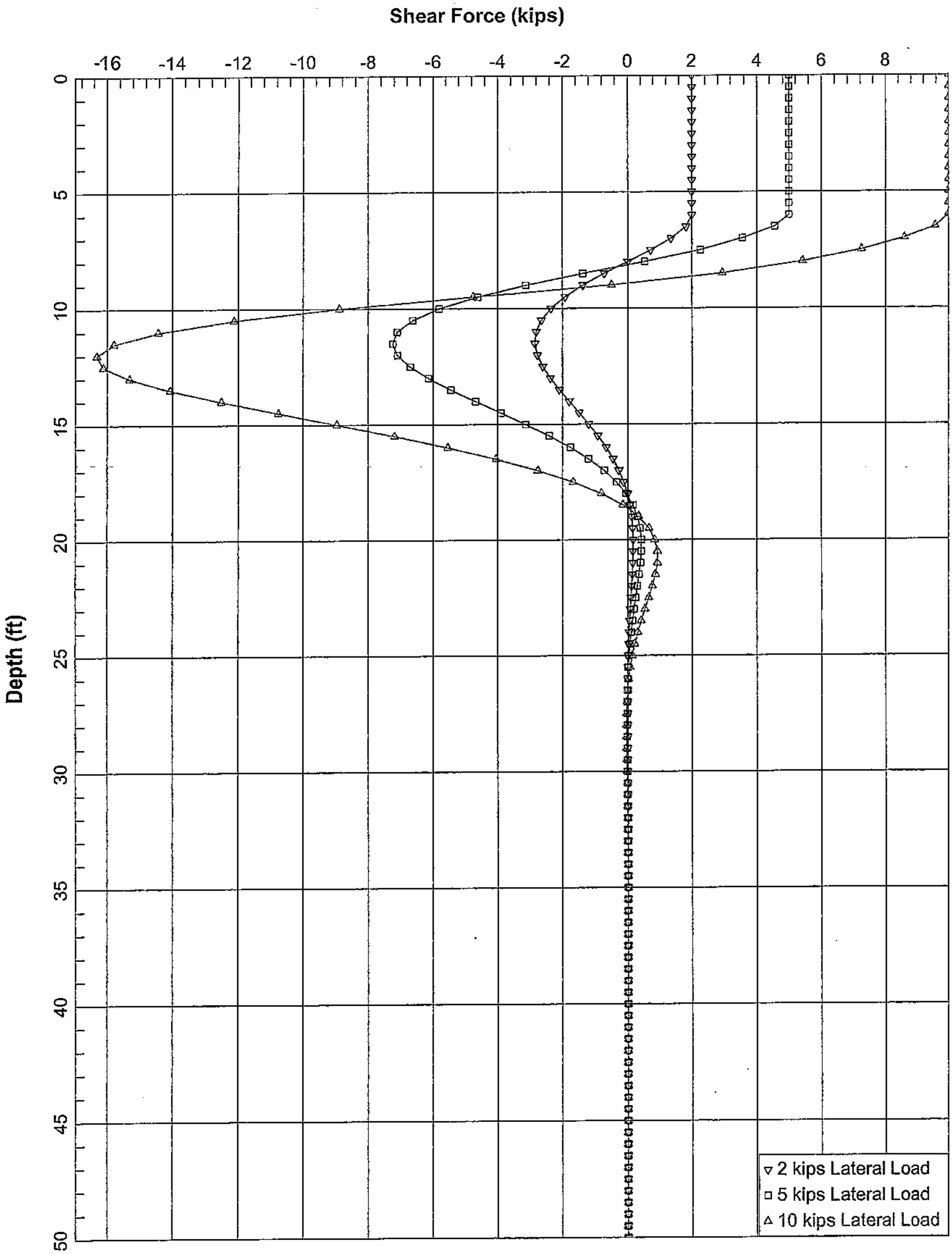
18-inch Diameter Friction Pile, Free Head Condition
MGA NORTH, LLC - File No. 20572



18-inch Diameter Pile - Free Head - 75 kips Axial Load



18-inch Diameter Pile - Free Head - 75 kips Axial Load



18-inch Diameter Pile - Free Head - 200 kips Axial Load

LPILE Plus for Windows, Version 4.0 (4.0.8)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

This program is licensed to:

Geotechnologies, Inc.
Geotechnologies, Inc.

Path to file locations:	F:\Gregorio\Pile Capacity\20572\
Name of input data file:	20572.18.Free.(NON-BEARING SOILS = 6FT).lpd
Name of output file:	20572.18.Free.(NON-BEARING SOILS = 6FT).lpo
Name of plot output file:	20572.18.Free.(NON-BEARING SOILS = 6FT).lpp
Name of runtime file:	20572.18.Free.(NON-BEARING SOILS = 6FT).lpr

Time and Date of Analysis

Date: August 14, 2013 Time: 10:26:58

Problem Title

MGA North, LLC - File No. 20572

Program Options

Units Used in Computations - US Customary Units, inches, pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- No computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

20572.18.Free.(NON-BEARING SOILS = 6FT).lpo

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 600.00 in
Depth of ground surface below top of pile = 72.00 in
Slope angle of ground surface = .00 deg.

Structural properties of pile defined using 2 points

Point	Depth X in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	18.000	5153.0000	254.0000	3605000.000
2	600.0000	18.000	5153.0000	254.0000	3605000.000

Soil and Rock Layering Information

The soil profile is modelled using 1 layers

Layer 1 is silt with cohesion and friction
Distance from top of pile to top of layer = 72.000 in
Distance from top of pile to bottom of layer = 840.000 in
p-y subgrade modulus k for top of soil layer = 500.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 500.000 lbs/in**3

(Depth of lowest layer extends 240.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Distribution of effective unit weight of soil with depth is defined using 2 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	72.00	.06944
2	840.00	.06944

Shear Strength of Soils

Distribution of shear strength parameters with depth defined using 2 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %

1	72.000	1.04166	28.00	.00600	.0
2	840.000	1.04166	28.00	.00600	.0

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves

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Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 3

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 2000.000 lbs
Bending moment at pile head = .000 in-lbs
Axial load at pile head = 75000.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 5000.000 lbs
Bending moment at pile head = .000 in-lbs
Axial load at pile head = 75000.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Load Case Number 3

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 10000.000 lbs
Bending moment at pile head = .000 in-lbs
Axial load at pile head = 75000.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 2000.000 lbs
Specified bending moment at pile head = .000 in-lbs
Specified axial load at pile head = 75000.000 lbs

(Zero moment for this load indicates free-head conditions)

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Soil Res p lbs/in
0.000	.083873	1.217E-07	2000.0000	-9.377E-04	295.2756	0.0000
6.000	.078247	12421.9509	2000.0000	-9.357E-04	316.9712	0.0000
12.000	.072645	24842.0964	2000.0000	-9.296E-04	338.6637	0.0000
18.000	.067091	37258.6312	2000.0000	-9.196E-04	360.3499	0.0000
24.000	.061610	49669.7507	2000.0000	-9.056E-04	382.0266	0.0000
30.000	.056224	62073.6509	2000.0000	-8.875E-04	403.6907	0.0000
36.000	.050959	74468.5291	2000.0000	-8.655E-04	425.3390	0.0000
42.000	.045839	86852.5838	2000.0000	-8.394E-04	446.9684	0.0000
48.000	.040886	99224.0148	2000.0000	-8.094E-04	468.5758	0.0000
54.000	.036126	111581.0242	2000.0000	-7.753E-04	490.1580	0.0000
60.000	.031582	123921.8160	2000.0000	-7.373E-04	511.7119	0.0000
66.000	.027278	136244.5964	2000.0000	-6.953E-04	533.2343	0.0000
72.000	.023239	148547.5744	2000.0000	-6.493E-04	554.7222	0.0000
78.000	.019487	160828.9618	1824.6181	-5.993E-04	576.1723	-58.4606
84.000	.016047	170982.3912	1360.3954	-5.457E-04	593.9058	-96.2803
90.000	.012938	177644.8785	722.2309	-4.894E-04	605.5422	-116.4412
96.000	.010173	180089.6634	6.6662	-4.317E-04	609.8122	-122.0804
102.000	.007758	178113.3794	-708.6769	-3.738E-04	606.3605	-116.3673
108.000	.005687	171921.9840	-1364.9011	-3.173E-04	595.5469	-102.3741
114.000	.003950	162020.1346	-1920.8888	-2.634E-04	578.2527	-82.9552
120.000	.002527	149108.3501	-2351.7003	-2.131E-04	555.7016	-60.6487
126.000	.001393	133991.5415	-2646.4607	-1.674E-04	529.2992	-37.6048
132.000	5.18E-04	117501.4853	-2805.9111	-1.268E-04	500.4984	-15.5453
138.000	-1.29E-04	100434.7184	-2839.8054	-9.159E-05	470.6904	4.2473
144.000	-5.81E-04	83506.2551	-2764.3208	-6.189E-05	441.1239	20.9143
150.000	-8.71E-04	67318.5691	-2599.6274	-3.753E-05	412.8512	33.9835
156.000	-.001031	52344.5044	-2367.7291	-1.821E-05	386.6982	43.3159
162.000	-.001090	38922.2059	-2090.6511	-3.468E-06	363.2554	49.0434
168.000	-.001073	27259.8121	-1789.0168	7.220E-06	342.8864	51.5014
174.000	-.001003	17447.5061	-1481.0215	1.444E-05	325.7486	51.1637
180.000	-9.00E-04	9474.5586	-1181.7847	1.879E-05	311.8234	48.5818
186.000	-7.78E-04	3249.1800	-903.0430	2.084E-05	300.9505	44.3321
192.000	-6.50E-04	-1380.7163	-653.1276	2.114E-05	297.6871	38.9731
198.000	-5.24E-04	-4607.3817	-437.1683	2.018E-05	303.3226	33.0134
204.000	-4.07E-04	-6644.8953	-257.4585	1.836E-05	306.8813	26.8899
210.000	-3.04E-04	-7713.4083	-113.9232	1.604E-05	308.7475	20.9552
216.000	-2.15E-04	-8026.4113	-4.6338	1.350E-05	309.2942	15.4746
222.000	-1.42E-04	-7781.1636	73.6733	1.095E-05	308.8658	10.6278
228.000	-8.36E-05	-7152.1836	125.1106	8.535E-06	307.7673	6.5179
234.000	-3.93E-05	-6287.5182	154.2101	6.365E-06	306.2571	3.1819
240.000	-7.19E-06	-5307.3902	165.5670	4.492E-06	304.5452	.6037
246.000	1.46E-05	-4304.7569	163.5615	2.940E-06	302.7941	-1.2722
252.000	2.81E-05	-3347.2983	152.1601	1.704E-06	301.1218	-2.5282
258.000	3.51E-05	-2480.3694	134.7901	7.630E-07	299.6077	-3.2618
264.000	3.72E-05	-1730.5036	114.2776	8.296E-08	298.2980	-3.5757
270.000	3.61E-05	-1109.1130	92.8381	-3.756E-07	297.2127	-3.5708
276.000	3.27E-05	-616.1083	72.1075	-6.542E-07	296.3517	-3.3395
282.000	2.82E-05	-243.2346	53.2006	-7.930E-07	295.7004	-2.9628
288.000	2.32E-05	23.0126	36.7877	-8.286E-07	295.3158	-2.5082
294.000	1.83E-05	198.9629	23.1778	-7.927E-07	295.6231	-2.0285
300.000	1.37E-05	301.8596	12.4033	-7.119E-07	295.8028	-1.5630
306.000	9.73E-06	348.4431	4.2982	-6.068E-07	295.8842	-1.1387
312.000	6.43E-06	353.9836	-1.4323	-4.934E-07	295.8938	-.7715
318.000	3.81E-06	331.7000	-5.1531	-3.827E-07	295.8549	-.4688
324.000	1.84E-06	292.4908	-7.2539	-2.819E-07	295.7864	-.2315
330.000	4.29E-07	244.9066	-8.1144	-1.951E-07	295.7033	-.055370
336.000	-5.04E-07	195.2936	-8.0810	-1.240E-07	295.6167	.066515
342.000	-1.06E-06	148.0466	-7.4527	-6.853E-08	295.5342	.1429
348.000	-1.33E-06	105.9228	-6.4749	-2.752E-08	295.4606	.1830
354.000	-1.39E-06	70.3726	-5.3383	9.509E-10	295.3985	.1958

360.000	-1.31E-06	41.8617	-4.1829	1.908E-08	295.3487	.1893
366.000	-1.16E-06	20.1612	-3.1033	2.909E-08	295.3108	.1705
372.000	-9.66E-07	4.5959	-2.1572	3.309E-08	295.2836	.1449
378.000	-7.63E-07	-5.7549	-1.3725	3.290E-08	295.2856	.1167
384.000	-5.71E-07	-11.9034	-.7551	3.005E-08	295.2964	.089068
390.000	-4.02E-07	-14.8436	-.2961	2.573E-08	295.3015	.063946
396.000	-2.62E-07	-15.4797	.023155	2.084E-08	295.3026	.042471
402.000	-1.52E-07	-14.5845	.2259	1.598E-08	295.3011	.025106
408.000	-7.04E-08	-12.7834	.3367	1.156E-08	295.2979	.011828
414.000	-1.34E-08	-10.5547	.3791	7.791E-09	295.2940	.002297
420.000	2.31E-08	-8.2417	.3739	4.756E-09	295.2900	-.004017
426.000	4.36E-08	-6.0721	.3387	2.444E-09	295.2862	-.007723
432.000	5.24E-08	-4.1797	.2872	7.885E-10	295.2829	-.009435
438.000	5.31E-08	-2.6263	.2298	-3.106E-10	295.2802	-.009717
444.000	4.87E-08	-1.4224	.1734	-9.645E-10	295.2781	-.009056
450.000	4.15E-08	-.5443	.1227	-1.282E-09	295.2765	-.007848
456.000	3.33E-08	.051427	.079995	-1.362E-09	295.2757	-.006394
462.000	2.52E-08	.4169	.046080	-1.286E-09	295.2763	-.004911
468.000	1.79E-08	.6055	.020732	-1.121E-09	295.2766	-.003538
474.000	1.17E-08	.6667	.003042	-9.154E-10	295.2768	-.002358
480.000	6.89E-09	.6429	-.008247	-7.040E-10	295.2767	-.001405
486.000	3.28E-09	.5684	-.014501	-5.083E-10	295.2766	-6.800E-04
492.000	7.86E-10	.4693	-.017036	-3.408E-10	295.2764	-1.650E-04
498.000	-8.04E-10	.3643	-.017017	-2.061E-10	295.2762	1.713E-04
504.000	-1.69E-09	.2653	-.015409	-1.045E-10	295.2761	3.646E-04
510.000	-2.06E-09	.1795	-.012963	-3.264E-11	295.2759	4.507E-04
516.000	-2.08E-09	.1098	-.010226	1.407E-11	295.2758	4.617E-04
522.000	-1.89E-09	.056725	-.007566	4.096E-11	295.2757	4.250E-04
528.000	-1.59E-09	.018957	-.005205	5.318E-11	295.2756	3.621E-04
534.000	-1.25E-09	-.005778	-.003251	5.531E-11	295.2756	2.889E-04
540.000	-9.24E-10	-.020111	-.001736	5.113E-11	295.2756	2.163E-04
546.000	-6.37E-10	-.026652	-6.337E-04	4.358E-11	295.2756	1.510E-04
552.000	-4.02E-10	-.027754	1.085E-04	3.479E-11	295.2756	9.638E-05
558.000	-2.20E-10	-.025382	5.578E-04	2.621E-11	295.2756	5.339E-05
564.000	-8.70E-11	-.021084	7.822E-04	1.871E-11	295.2756	2.141E-05
570.000	4.76E-12	-.016012	8.429E-04	1.272E-11	295.2756	-1.186E-06
576.000	6.55E-11	-.010981	7.898E-04	8.356E-12	295.2756	-1.652E-05
582.000	1.05E-10	-.006542	6.599E-04	5.526E-12	295.2756	-2.678E-05
588.000	1.32E-10	-.003067	4.775E-04	3.975E-12	295.2756	-3.402E-05
594.000	1.53E-10	-8.160E-04	2.558E-04	3.347E-12	295.2756	-3.986E-05
600.000	1.72E-10	0.0000	0.0000	3.216E-12	295.2756	-4.541E-05

Output Verification:

Computed forces and moments are within specified convergence limits.

Output Summary for Load Case No. 1:

```
Pile-head deflection              =      .08387303 in
Computed slope at pile head       =     -.00093767
Maximum bending moment            =   180089.663 lbs-in
Maximum shear force               =    -2839.805 lbs
Depth of maximum bending moment   =       96.000 in
Depth of maximum shear force      =      138.000 in
Number of iterations              =            5
Number of zero deflection points  =            6
```

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 2

```
Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head    =        5000.000 lbs
Specified bending moment at pile head =            .000 in-lbs
Specified axial load at pile head     =       75000.000 lbs
```

(Zero moment for this load indicates free-head conditions)

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Soil Res p lbs/in
0.000	.215580	-1.719E-07	5000.0000	-.002390	295.2756	0.0000
6.000	.201242	31075.3031	5000.0000	-.002385	349.5503	0.0000
12.000	.186965	62146.0896	5000.0000	-.002369	403.8172	0.0000
18.000	.172809	93207.8436	5000.0000	-.002344	458.0683	0.0000
24.000	.158833	124256.0503	5000.0000	-.002309	512.2957	0.0000
30.000	.145097	155286.1970	5000.0000	-.002264	566.4915	0.0000
36.000	.131663	186293.7738	5000.0000	-.002209	620.6480	0.0000
42.000	.118590	217274.2738	5000.0000	-.002144	674.7571	0.0000
48.000	.105937	248223.1943	5000.0000	-.002069	728.8112	0.0000
54.000	.093766	279136.0368	5000.0000	-.001983	782.8021	0.0000
60.000	.082136	310008.3085	5000.0000	-.001888	836.7223	0.0000
66.000	.071106	340835.5222	5000.0000	-.001783	890.5637	0.0000
72.000	.060737	371613.1974	5000.0000	-.001668	944.3186	0.0000
78.000	.051088	402336.8607	4558.5158	-.001543	997.9792	-147.1614
84.000	.042219	427704.2361	3560.8387	-.001409	1042.2847	-185.3976
90.000	.034179	446335.1326	2257.3350	-.001268	1074.8246	-249.1036
96.000	.027004	455933.4269	537.8940	-.001122	1091.5886	-324.0434
102.000	.020712	453799.8918	-1366.2698	-9.753E-04	1087.8622	-310.6779
108.000	.015300	440415.9954	-3124.4783	-8.309E-04	1064.4865	-275.3916
114.000	.010741	417053.9895	-4627.3172	-6.925E-04	1023.6835	-225.5547
120.000	.006990	385511.3976	-5807.2676	-5.628E-04	968.5926	-167.7621
126.000	.003987	347873.3385	-6633.4657	-4.444E-04	902.8556	-107.6373
132.000	.001657	306309.7754	-7105.5255	-3.388E-04	830.2626	-49.7159
138.000	-7.86E-05	262911.9177	-7246.8954	-2.468E-04	754.4658	2.5926
144.000	-.001305	219569.1819	-7098.1966	-1.689E-04	678.7653	46.9736
150.000	-.002106	177885.5839	-6710.9236	-1.047E-04	605.9626	82.1174
156.000	-.002562	139132.3563	-6141.8106	-5.353E-05	538.2780	107.5870
162.000	-.002748	104232.0378	-5448.0713	-1.423E-05	477.3226	123.6594
168.000	-.002732	73768.3097	-4683.6293	1.451E-05	424.1160	131.1546
174.000	-.002574	48015.4241	-3896.3699	3.418E-05	379.1372	131.2652
180.000	-.002322	26981.1084	-3126.3746	4.629E-05	342.3996	125.3999
186.000	-.002018	10457.2661	-2405.0417	5.234E-05	313.5398	115.0444
192.000	-.001694	-1926.4964	-1754.9592	5.372E-05	298.6403	101.6497
198.000	-.001374	-10650.5890	-1190.3744	5.168E-05	313.8774	86.5452
204.000	-.001074	-16257.5053	-718.0974	4.734E-05	323.6702	70.8804
210.000	-8.06E-04	-19310.3634	-338.6838	4.160E-05	329.0022	55.5908
216.000	-5.75E-04	-20359.1466	-47.7539	3.519E-05	330.8340	41.3858
222.000	-3.83E-04	-19915.0804	162.6681	2.868E-05	330.0584	28.7548
228.000	-2.31E-04	-18432.9457	302.8897	2.249E-05	327.4698	17.9857
234.000	-1.13E-04	-16300.6464	384.4264	1.688E-05	323.7456	9.1932
240.000	-2.80E-05	-13835.0238	419.0607	1.202E-05	319.4392	2.3516
246.000	3.07E-05	-11282.7324	418.1044	7.959E-06	314.9815	-2.6704
252.000	6.75E-05	-8824.9348	391.8633	4.712E-06	310.6888	-6.0766
258.000	8.72E-05	-6584.6138	349.2933	2.224E-06	306.7760	-8.1134
264.000	9.42E-05	-4635.4169	297.8229	4.117E-07	303.3716	-9.0434
270.000	9.22E-05	-3011.1090	243.3150	-8.232E-07	300.5347	-9.1259
276.000	8.43E-05	-1714.8960	190.1341	-1.586E-06	298.2708	-8.6011
282.000	7.31E-05	-728.0718	141.2905	-1.981E-06	296.5472	-7.6801
288.000	6.06E-05	-17.6266	98.6309	-2.101E-06	295.3064	-6.5397
294.000	4.79E-05	457.3907	63.0517	-2.030E-06	296.0744	-5.3200
300.000	3.62E-05	740.8214	34.7150	-1.837E-06	296.5695	-4.1256
306.000	2.59E-05	875.6238	13.2522	-1.576E-06	296.8049	-3.0287
312.000	1.73E-05	901.2665	-2.0546	-1.289E-06	296.8497	-2.0736
318.000	1.04E-05	852.1288	-12.1205	-1.006E-06	296.7639	-1.2817
324.000	5.21E-06	756.7262	-17.9357	-7.458E-07	296.5973	-.6567

330.000	1.47E-06	637.5715	-20.4749	-5.207E-07	296.3891	-.1897
336.000	-1.04E-06	511.4961	-20.6337	-3.351E-07	296.1689	.1367
342.000	-2.55E-06	390.2688	-19.1904	-1.895E-07	295.9572	.3444
348.000	-3.31E-06	281.3821	-16.7871	-8.100E-08	295.7670	.4567
354.000	-3.52E-06	188.8960	-13.9268	-5.053E-09	295.6055	.4967
360.000	-3.37E-06	114.2646	-10.9808	4.391E-08	295.4752	.4853
366.000	-3.00E-06	57.0874	-8.2036	7.158E-08	295.3753	.4404
372.000	-2.51E-06	15.7567	-5.7523	8.334E-08	295.3031	.3767
378.000	-2.00E-06	-12.0157	-3.7062	8.395E-08	295.2966	.3054
384.000	-1.50E-06	-28.7929	-2.0862	7.736E-08	295.3259	.2346
390.000	-1.07E-06	-37.1203	-.8732	6.671E-08	295.3404	.1698
396.000	-7.03E-07	-39.3310	-.022084	5.436E-08	295.3443	.1139
402.000	-4.15E-07	-37.4342	.5253	4.197E-08	295.3410	.068519
408.000	-2.00E-07	-33.0652	.8315	3.058E-08	295.3333	.033556
414.000	-4.83E-08	-27.4835	.9570	2.080E-08	295.3236	.008256
420.000	4.99E-08	-21.6005	.9557	1.288E-08	295.3133	-.008684
426.000	1.06E-07	-16.0270	.8732	6.800E-09	295.3036	-.018804
432.000	1.32E-07	-11.1281	.7458	2.415E-09	295.2950	-.023672
438.000	1.35E-07	-7.0798	.6005	-5.255E-10	295.2880	-.024745
444.000	1.25E-07	-3.9213	.4564	-2.302E-09	295.2824	-.023288
450.000	1.08E-07	-1.6006	.3256	-3.194E-09	295.2784	-.020335
456.000	8.69E-08	-.011802	.2145	-3.454E-09	295.2756	-.016681
462.000	6.61E-08	.9765	.1258	-3.298E-09	295.2773	-.012898
468.000	4.73E-08	1.5004	.058978	-2.898E-09	295.2782	-.009365
474.000	3.14E-08	1.6869	.011971	-2.384E-09	295.2785	-.006304
480.000	1.87E-08	1.6462	-.018381	-1.845E-09	295.2785	-.003814
486.000	9.22E-09	1.4679	-.035545	-1.343E-09	295.2782	-.001908
492.000	2.58E-09	1.2208	-.042895	-9.084E-10	295.2777	-5.424E-04
498.000	-1.68E-09	.9540	-.043446	-5.571E-10	295.2773	3.587E-04
504.000	-4.10E-09	.7000	-.039711	-2.900E-10	295.2768	8.862E-04
510.000	-5.16E-09	.4778	-.033660	-9.981E-11	295.2764	.001131
516.000	-5.30E-09	.2962	-.026737	2.517E-11	295.2761	.001177
522.000	-4.86E-09	.1569	-.019925	9.834E-11	295.2759	.001094
528.000	-4.12E-09	.056981	-.013824	1.329E-10	295.2757	9.394E-04
534.000	-3.27E-09	-.009119	-.008741	1.406E-10	295.2756	7.549E-04
540.000	-2.43E-09	-.048042	-.004769	1.314E-10	295.2757	5.693E-04
546.000	-1.69E-09	-.066463	-.001858	1.129E-10	295.2757	4.008E-04
552.000	-1.08E-09	-.070443	1.208E-04	9.077E-11	295.2757	2.588E-04
558.000	-6.02E-10	-.065095	.001336	6.888E-11	295.2757	1.463E-04
564.000	-2.52E-10	-.054470	.001961	4.957E-11	295.2757	6.195E-05
570.000	-7.18E-12	-.041608	.002152	3.406E-11	295.2757	1.787E-06
576.000	1.57E-10	-.028675	.002039	2.271E-11	295.2756	-3.953E-05
582.000	2.65E-10	-.017160	.001717	1.531E-11	295.2756	-6.766E-05
588.000	3.41E-10	-.008079	.001251	1.123E-11	295.2756	-8.786E-05
594.000	4.00E-10	-.002160	6.740E-04	9.577E-12	295.2756	-1.044E-04
600.000	4.55E-10	0.0000	0.0000	9.228E-12	295.2756	-1.202E-04

Output Verification:

Computed forces and moments are within specified convergence limits.

Output Summary for Load Case No. 2:

Pile-head deflection	=	.21557984 in
Computed slope at pile head	=	-.00238956
Maximum bending moment	=	455933.427 lbs-in
Maximum shear force	=	-7246.895 lbs
Depth of maximum bending moment	=	96.000 in
Depth of maximum shear force	=	138.000 in
Number of iterations	=	8
Number of zero deflection points	=	6

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 3

Pile-head boundary conditions are Shear and Moment (BC Type 1)

Specified shear force at pile head	=	10000.000 lbs
Specified bending moment at pile head	=	.000 in-lbs
Specified axial load at pile head	=	75000.000 lbs

(Zero moment for this load indicates free-head conditions)

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Soil Res p lbs/in
0.000	.516761	-3.437E-07	10000.0000	-.005414	295.2756	0.0000
6.000	.484276	62436.3531	10000.0000	-.005404	404.3241	0.0000
12.000	.451912	124863.6314	10000.0000	-.005374	513.3568	0.0000
18.000	.419790	187272.7614	10000.0000	-.005323	622.3578	0.0000
24.000	.388032	249654.6725	10000.0000	-.005253	731.3113	0.0000
30.000	.356757	312000.2976	10000.0000	-.005162	840.2014	0.0000
36.000	.326086	374300.5752	10000.0000	-.005051	949.0123	0.0000
42.000	.296141	436546.4503	10000.0000	-.004920	1057.7282	0.0000
48.000	.267042	498728.8758	10000.0000	-.004769	1166.3332	0.0000
54.000	.238910	560838.8138	10000.0000	-.004598	1274.8117	0.0000
60.000	.211864	622867.2371	10000.0000	-.004407	1383.1477	0.0000
66.000	.186026	684805.1301	10000.0000	-.004196	1491.3257	0.0000
72.000	.161514	746643.4905	10000.0000	-.003965	1599.3298	0.0000
78.000	.138450	808373.3304	9582.2457	-.003714	1707.1444	-139.2514
84.000	.116952	864972.6262	8594.3638	-.003443	1805.9982	-190.0425
90.000	.097130	914604.6716	7250.9253	-.003156	1892.6833	-257.7703
96.000	.079081	954824.0542	5426.3310	-.002854	1962.9287	-350.4278
102.000	.062882	982289.2564	2937.5433	-.002541	2010.8982	-479.1681
108.000	.048586	992361.6385	-485.2886	-.002222	2028.4902	-661.7759
114.000	.036214	978465.8537	-4752.1111	-.001904	2004.2204	-760.4983
120.000	.025738	937049.9161	-8886.7582	-.001595	1931.8852	-617.7174
126.000	.017078	873259.9565	-12123.2431	-.001302	1820.4725	-461.1109
132.000	.010110	792743.0823	-14416.5160	-.001033	1679.8453	-303.3134
138.000	.004679	701191.7039	-15789.6765	-7.920E-04	1519.9458	-154.4067
144.000	6.06E-04	603979.7685	-16318.3872	-5.812E-04	1350.1597	-21.8302
150.000	-.002296	505894.1615	-16115.2766	-4.020E-04	1178.8478	89.5337
156.000	-.004217	410958.2397	-15315.2728	-2.539E-04	1013.0369	177.1342
162.000	-.005343	322339.4197	-14062.5893	-1.355E-04	858.2592	240.4269
168.000	-.005843	242329.1185	-12499.8461	-4.431E-05	718.5168	280.4875
174.000	-.005875	172381.1455	-10759.5787	2.266E-05	596.3488	299.6017
180.000	-.005572	113193.7777	-8958.1852	6.878E-05	492.9748	300.8628
186.000	-.005049	64821.0192	-7192.1898	9.753E-05	408.4891	287.8023
192.000	-.004401	26799.7235	-5536.5711	1.123E-04	342.0828	264.0705
198.000	-.003701	-1718.9280	-4044.8230	1.164E-04	298.2778	233.1789
204.000	-.003005	-21842.8909	-2750.3640	1.126E-04	333.4254	198.3075
210.000	-.002350	-34824.6097	-1668.9096	1.034E-04	356.0987	162.1773
216.000	-.001764	-41962.8838	-801.4356	9.102E-05	368.5661	126.9807
222.000	-.001258	-44523.7547	-137.4063	7.705E-05	373.0388	94.3624
228.000	-8.39E-04	-43681.1066	342.0058	6.281E-05	371.5671	65.4416
234.000	-5.04E-04	-40476.2115	660.9181	4.922E-05	365.9695	40.8625
240.000	-2.48E-04	-35794.3843	846.1010	3.690E-05	357.7925	20.8652
246.000	-6.17E-05	-30356.2087	924.7954	2.622E-05	348.2944	5.3663
252.000	6.62E-05	-24720.4345	923.0192	1.732E-05	338.4512	-5.9583
258.000	1.46E-04	-19295.5675	864.3593	1.021E-05	328.9764	-13.5950
264.000	1.89E-04	-14357.3155	769.2091	4.779E-06	320.3514	-18.1217
270.000	2.04E-04	-10069.3596	654.3954	8.342E-07	312.8623	-20.1495
276.000	1.99E-04	-6505.3220	533.1206	-1.842E-06	306.6375	-20.2754
282.000	1.81E-04	-3670.2544	415.1469	-3.486E-06	301.6859	-19.0491
288.000	1.57E-04	-1520.4221	307.1479	-4.324E-06	297.9311	-16.9505
294.000	1.30E-04	19.4125	213.1623	-4.566E-06	295.3095	-14.3780

300.000	1.02E-04	1041.6355	135.0923	-4.395E-06	297.0949	-11.6453
306.000	7.68E-05	1644.4760	73.2029	-3.961E-06	298.1478	-8.9845
312.000	5.46E-05	1923.6360	26.5877	-3.385E-06	298.6353	-6.5539
318.000	3.62E-05	1966.5749	-6.4206	-2.757E-06	298.7103	-4.4488
324.000	2.15E-05	1849.0694	-27.9071	-2.141E-06	298.5051	-2.7133
330.000	1.05E-05	1633.6168	-40.1034	-1.578E-06	298.1288	-1.3522
336.000	2.60E-06	1369.2490	-45.1877	-1.093E-06	297.6671	-.3426
342.000	-2.64E-06	1092.3479	-45.1475	-6.957E-07	297.1834	.3560
348.000	-5.75E-06	828.1050	-41.6976	-3.856E-07	296.7219	.7939
354.000	-7.26E-06	592.3233	-36.2431	-1.562E-07	296.3101	1.0243
360.000	-7.63E-06	393.3288	-29.8752	2.982E-09	295.9626	1.0984
366.000	-7.23E-06	233.8180	-23.3924	1.043E-07	295.6840	1.0626
372.000	-6.38E-06	112.5261	-17.3353	1.602E-07	295.4721	.9564
378.000	-5.31E-06	25.6501	-12.0305	1.825E-07	295.3204	.8118
384.000	-4.19E-06	-32.0036	-7.6358	1.815E-07	295.3315	.6530
390.000	-3.13E-06	-66.1429	-4.1844	1.656E-07	295.3911	.4974
396.000	-2.20E-06	-82.3660	-1.6237	1.417E-07	295.4194	.3562
402.000	-1.43E-06	-85.7549	.1519	1.145E-07	295.4254	.2357
408.000	-8.25E-07	-80.6459	1.2747	8.763E-08	295.4164	.1385
414.000	-3.77E-07	-70.5380	1.8837	6.321E-08	295.3988	.064467
420.000	-6.61E-08	-58.0989	2.1116	4.244E-08	295.3771	.011498
426.000	1.32E-07	-45.2375	2.0758	2.575E-08	295.3546	-.023409
432.000	2.43E-07	-33.2123	1.8744	1.308E-08	295.3336	-.043724
438.000	2.89E-07	-22.7563	1.5845	4.042E-09	295.3153	-.052926
444.000	2.91E-07	-14.2023	1.2631	-1.927E-09	295.3004	-.054203
450.000	2.66E-07	-7.5975	.9496	-5.448E-09	295.2889	-.050291
456.000	2.26E-07	-2.8021	.6685	-7.127E-09	295.2805	-.043400
462.000	1.81E-07	.4312	.4327	-7.510E-09	295.2763	-.035210
468.000	1.36E-07	2.3970	.2463	-7.053E-09	295.2798	-.026913
474.000	9.59E-08	3.3936	.1077	-6.118E-09	295.2815	-.019281
480.000	6.25E-08	3.6955	.011650	-4.973E-09	295.2820	-.012751
486.000	3.62E-08	3.5378	-.049115	-3.805E-09	295.2818	-.007503
492.000	1.68E-08	3.1095	-.082239	-2.732E-09	295.2810	-.003538
498.000	3.47E-09	2.5534	-.095070	-1.817E-09	295.2801	-7.392E-04
504.000	-4.96E-09	1.9703	-.094075	-1.086E-09	295.2790	.001071
510.000	-9.57E-09	1.4255	-.084577	-5.380E-10	295.2781	.002095
516.000	-1.14E-08	.9559	-.070690	-1.535E-10	295.2773	.002534
522.000	-1.14E-08	.5774	-.055388	9.415E-11	295.2766	.002567
528.000	-1.03E-08	.2911	-.040653	2.344E-10	295.2761	.002345
534.000	-8.60E-09	.089314	-.027661	2.958E-10	295.2757	.001986
540.000	-6.73E-09	-.041054	-.016978	3.036E-10	295.2757	.001576
546.000	-4.95E-09	-.1147	-.008730	2.785E-10	295.2758	.001174
552.000	-3.39E-09	-.1461	-.002766	2.364E-10	295.2758	8.141E-04
558.000	-2.12E-09	-.1481	.001218	1.889E-10	295.2758	5.140E-04
564.000	-1.13E-09	-.1316	.003590	1.437E-10	295.2758	2.769E-04
570.000	-3.91E-10	-.1051	.004713	1.055E-10	295.2758	9.731E-05
576.000	1.40E-10	-.075153	.004899	7.635E-11	295.2757	-3.531E-05
582.000	5.25E-10	-.046424	.004391	5.672E-11	295.2757	-1.340E-04
588.000	8.21E-10	-.022510	.003354	4.559E-11	295.2756	-2.118E-04
594.000	1.07E-09	-.006217	.001879	4.095E-11	295.2756	-2.799E-04
600.000	1.31E-09	0.0000	0.0000	3.994E-11	295.2756	-3.464E-04

Output Verification:

Computed forces and moments are within specified convergence limits.

Output Summary for Load Case No. 3:

```
Pile-head deflection            =      .51676060 in
Computed slope at pile head     =    -.00541412
Maximum bending moment          =    992361.639 lbs-in
Maximum shear force             =    -16318.387 lbs
Depth of maximum bending moment =       108.000 in
Depth of maximum shear force    =       144.000 in
Number of iterations            =            13
```


Number of zero deflection points = 6

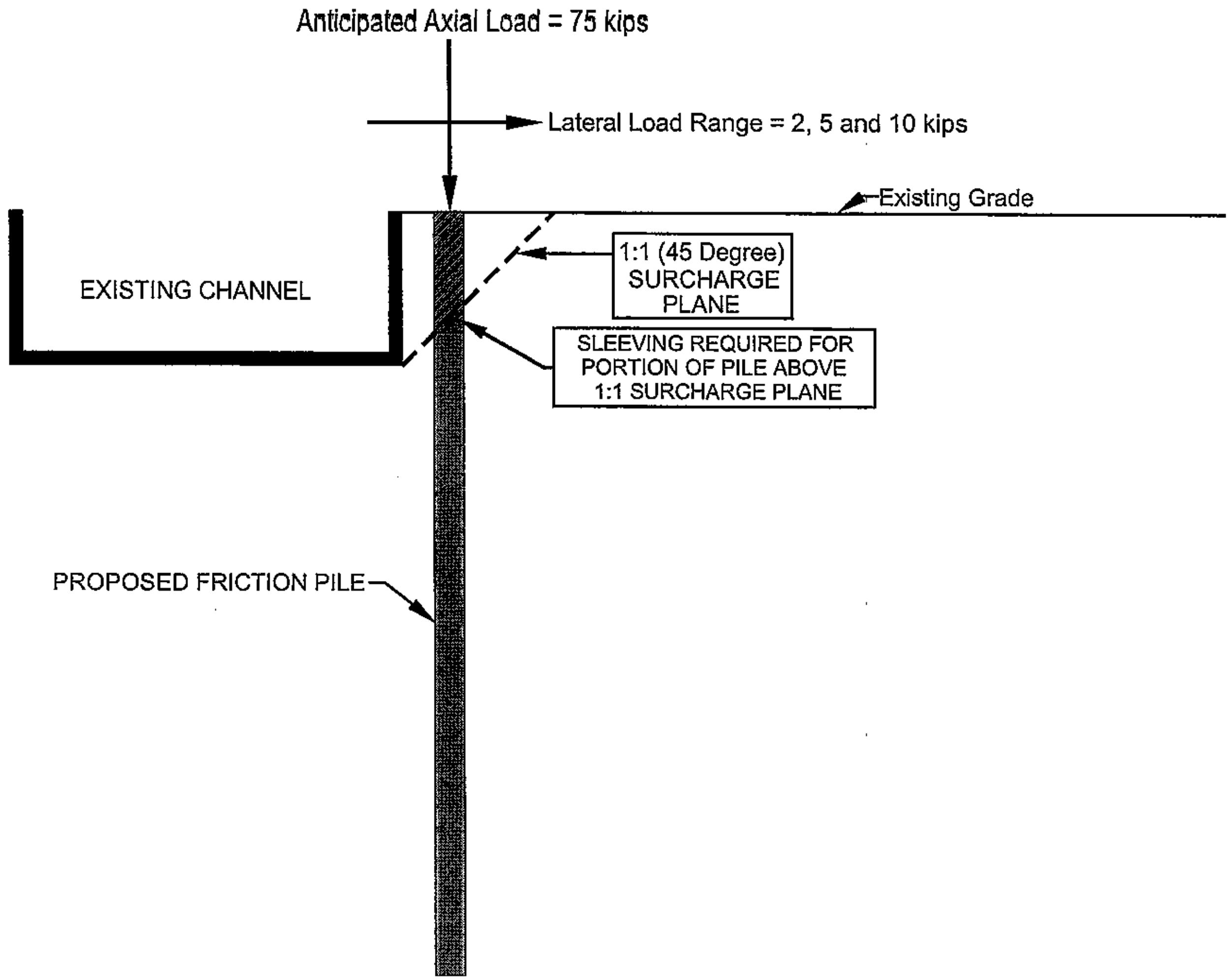
Summary of Pile-head Response

Definition of symbols for pile-head boundary conditions:

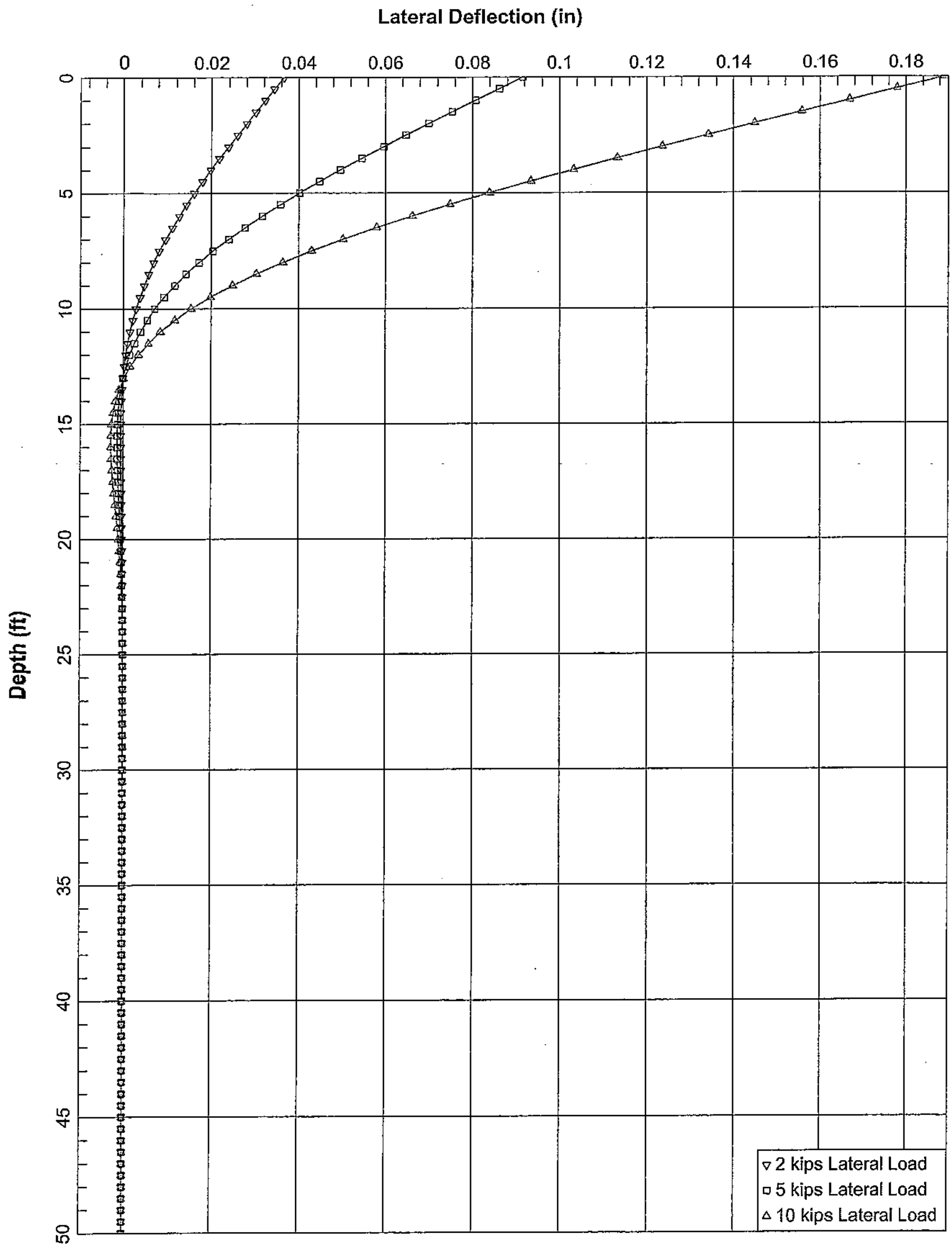
y = pile-head displacment, in
M = pile-head moment, lbs-in
V = pile-head shear force, lbs
S = pile-head slope, radians
R = rotational stiffness of pile-head, in-lbs/rad

BC Type	Boundary Condition 1	Boundary Condition 2	Axial Load lbs	Pile Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
1	V= 2000.000	M= 0.000	75000.0000	.083873	180089.6634	-2839.8054
1	V= 5000.000	M= 0.000	75000.0000	.2156	455933.4269	-7246.8954
1	V= 10000.000	M= 0.000	75000.0000	.5168	992361.6385	-16318.3872

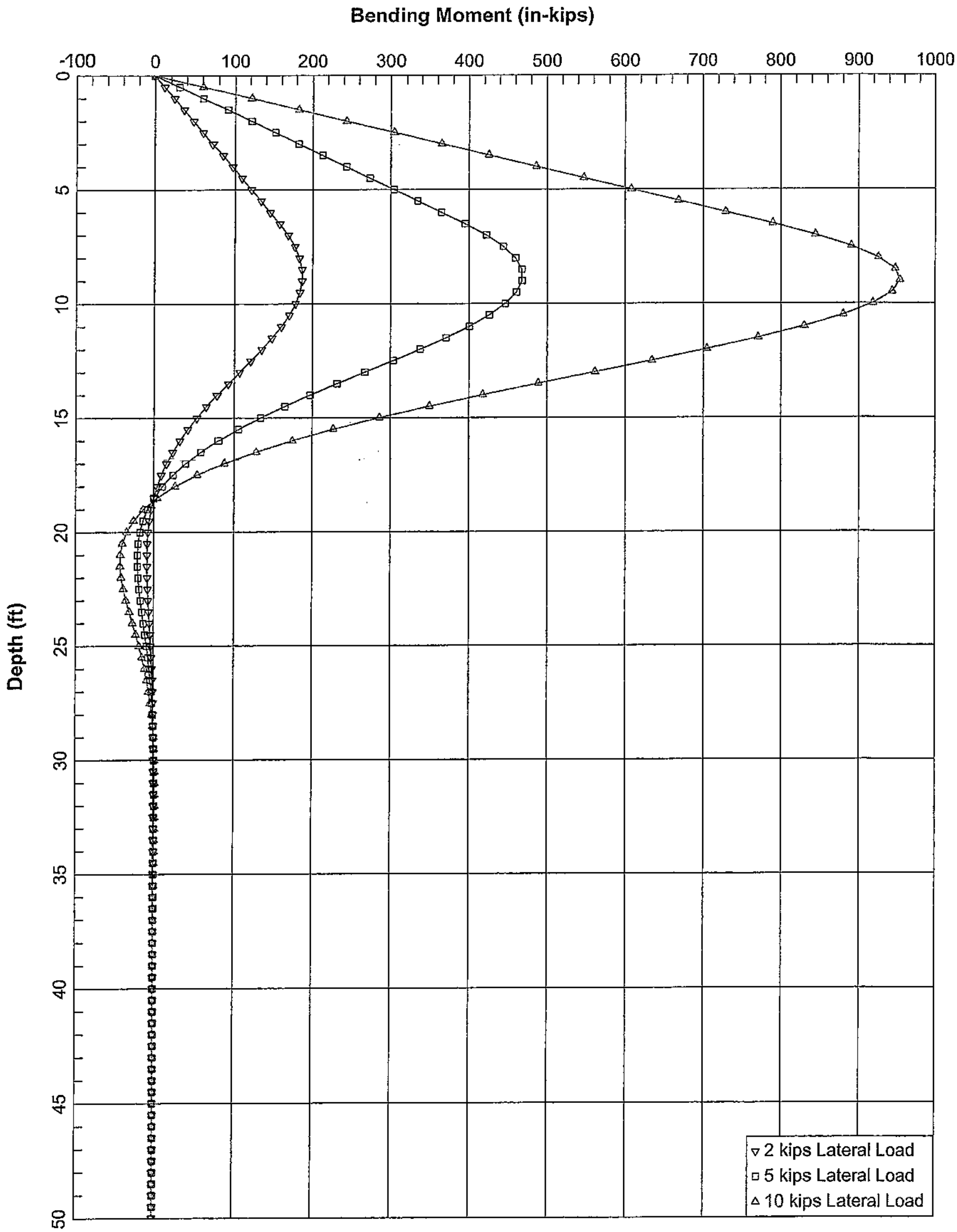
The analysis ended normally.



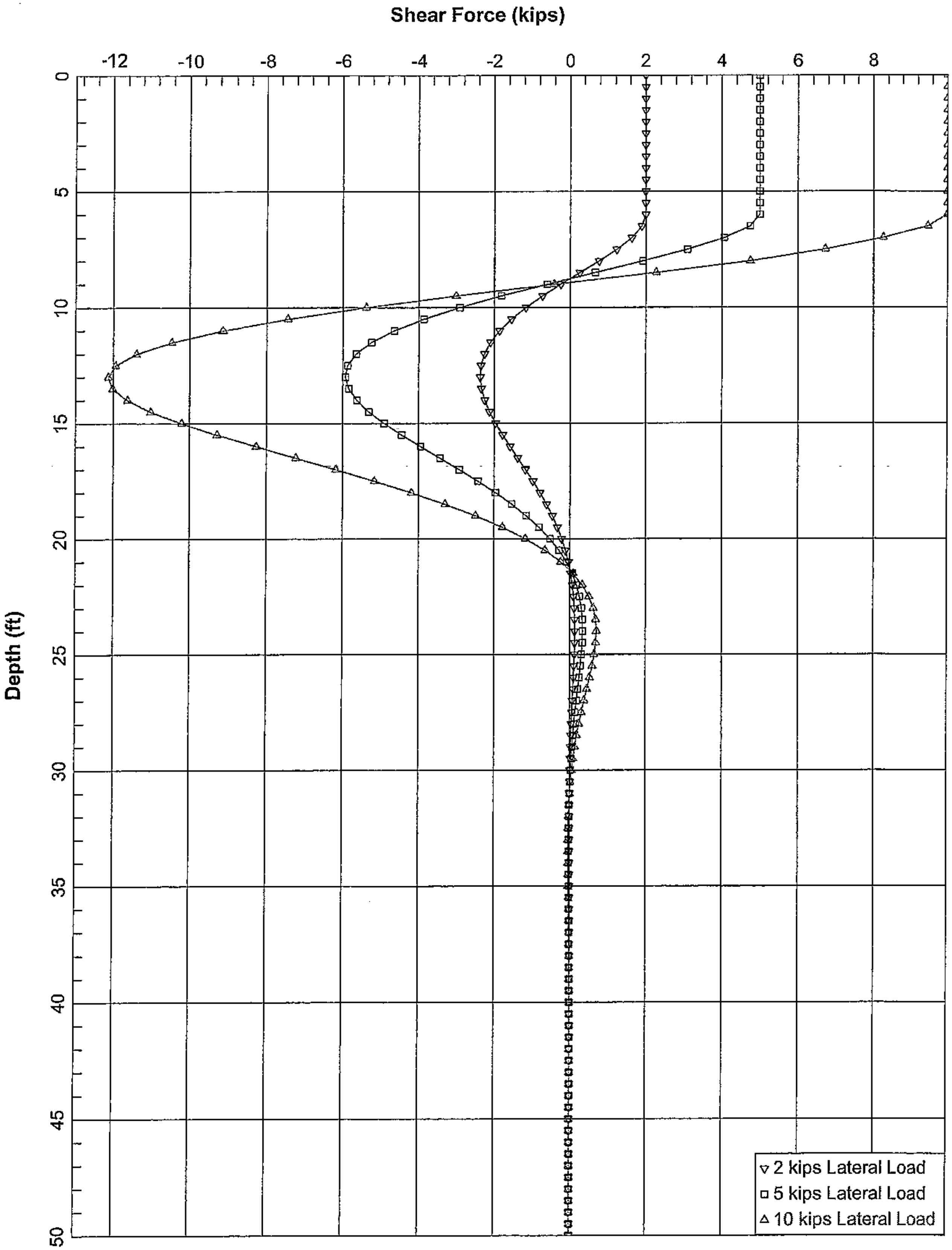
24-inch Diameter Friction Pile, Free Head Condition
MGA NORTH, LLC - File No. 20572



24-inch Diameter Pile - Free Head - 75 kips Axial Load



24-inch Diameter Pile - Free Head - 75 kips Axial Load



24-inch Diameter Pile - Free Head - 200 kips Axial Load

LPILE Plus for Windows, Version 4.0 (4.0.8)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

This program is licensed to:

Geotechnologies, Inc.
Geotechnologies, Inc.

```
Path to file locations:     F:\Gregorio\Pile Capacity\20572\
Name of input data file:    20572.24.Free.(NON-BEARING SOILS = 6FT).lpd
Name of output file:        20572.24.Free.(NON-BEARING SOILS = 6FT).lpo
Name of plot output file:   20572.24.Free.(NON-BEARING SOILS = 6FT).lpp
Name of runtime file:       20572.24.Free.(NON-BEARING SOILS = 6FT).lpr
```

Time and Date of Analysis

Date: August 14, 2013   Time: 10:35:47

Problem Title

MGA North, LLC - File No. 20572

Program Options

Units Used in Computations - US Customary Units, inches, pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- No computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

```
Pile Length                              =   600.00 in
Depth of ground surface below top of pile =   72.00 in
Slope angle of ground surface            =      .00 deg.
```

Structural properties of pile defined using 2 points

Point	Depth X in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	24.000	16286.0000	452.0000	3605000.000
2	600.0000	24.000	16286.0000	452.0000	3605000.000

Soil and Rock Layering Information

The soil profile is modelled using 1 layers

```
Layer  1 is silt with cohesion and friction
Distance from top of pile to top of layer    =   72.000 in
Distance from top of pile to bottom of layer =  840.000 in
p-y subgrade modulus k for top of soil layer =  500.000 lbs/in**3
p-y subgrade modulus k for bottom of layer   =  500.000 lbs/in**3
```

(Depth of lowest layer extends 240.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Distribution of effective unit weight of soil with depth is defined using 2 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	72.00	.06944
2	840.00	.06944

Shear Strength of Soils

Distribution of shear strength parameters with depth defined using 2 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %

1	72.000	1.04166	28.00	.00600	.0
2	840.000	1.04166	28.00	.00600	.0

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves

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Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 3

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 2000.000 lbs
Bending moment at pile head = .000 in-lbs
Axial load at pile head = 75000.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 5000.000 lbs
Bending moment at pile head = .000 in-lbs
Axial load at pile head = 75000.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Load Case Number 3

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 10000.000 lbs
Bending moment at pile head = .000 in-lbs
Axial load at pile head = 75000.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 2000.000 lbs
Specified bending moment at pile head = .000 in-lbs
Specified axial load at pile head = 75000.000 lbs

(Zero moment for this load indicates free-head conditions)

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Soil Res p lbs/in
0.000	.036632	-3.395E-08	2000.0000	-3.628E-04	165.9292	0.0000
6.000	.034455	12163.2417	2000.0000	-3.621E-04	174.8914	0.0000
12.000	.032286	24325.9241	2000.0000	-3.603E-04	183.8533	0.0000
18.000	.030132	36487.4877	2000.0000	-3.572E-04	192.8142	0.0000
24.000	.028000	48647.3734	2000.0000	-3.528E-04	201.7740	0.0000
30.000	.025898	60805.0219	2000.0000	-3.472E-04	210.7321	0.0000
36.000	.023834	72959.8741	2000.0000	-3.404E-04	219.6882	0.0000
42.000	.021814	85111.3710	2000.0000	-3.323E-04	228.6417	0.0000
48.000	.019846	97258.9538	2000.0000	-3.230E-04	237.5924	0.0000
54.000	.017938	109402.0638	2000.0000	-3.124E-04	246.5398	0.0000
60.000	.016097	121540.1427	2000.0000	-3.006E-04	255.4835	0.0000
66.000	.014330	133672.6322	2000.0000	-2.876E-04	264.4231	0.0000
72.000	.012646	145798.9744	2000.0000	-2.733E-04	273.3581	0.0000
78.000	.011051	157918.6116	1900.5454	-2.578E-04	282.2882	-33.1515
84.000	.009552	168837.5308	1629.1514	-2.411E-04	290.3336	-57.3131
90.000	.008157	177685.4138	1236.9629	-2.234E-04	296.8530	-73.4164
96.000	.006872	183882.1344	769.3386	-2.049E-04	301.4189	-82.4583
102.000	.005698	187101.8991	265.5345	-1.860E-04	303.7913	-85.4764
108.000	.004640	187235.9094	-241.4574	-1.668E-04	303.8901	-83.5209
114.000	.003696	184354.5565	-724.8985	-1.478E-04	301.7670	-77.6261
120.000	.002866	178670.1844	-1164.1258	-1.293E-04	297.5786	-68.7830
126.000	.002145	170501.4089	-1544.2186	-1.114E-04	291.5596	-57.9146
132.000	.001529	160239.8658	-1855.5331	-9.455E-05	283.9986	-45.8569
138.000	.001010	148320.1061	-2093.1327	-7.878E-05	275.2158	-33.3430
144.000	5.83E-04	135193.1780	-2256.1440	-6.430E-05	265.5435	-20.9941
150.000	2.39E-04	121304.2440	-2347.0711	-5.119E-05	255.3097	-9.3149
156.000	-3.11E-05	107074.3947	-2371.0970	-3.952E-05	244.8247	1.3063
162.000	-2.35E-04	92886.6476	-2335.4001	-2.930E-05	234.3708	10.5927
168.000	-3.83E-04	79075.9656	-2248.5091	-2.052E-05	224.1947	18.3710
174.000	-4.82E-04	65923.0017	-2119.7151	-1.311E-05	214.5032	24.5604
180.000	-5.40E-04	53651.1804	-1958.5535	-6.996E-06	205.4610	29.1601
186.000	-5.66E-04	42426.6564	-1774.3673	-2.087E-06	197.1904	32.2353
192.000	-5.65E-04	32360.6506	-1575.9536	1.735E-06	189.7735	33.9027
198.000	-5.45E-04	23513.6526	-1371.2943	4.590E-06	183.2547	34.3171
204.000	-5.10E-04	15900.9879	-1167.3694	6.604E-06	177.6455	33.6579
210.000	-4.65E-04	9499.2769	-970.0430	7.902E-06	172.9285	32.1175
216.000	-4.15E-04	4253.3600	-784.0181	8.604E-06	169.0632	29.8908
222.000	-3.62E-04	83.3155	-612.8463	8.826E-06	165.9906	27.1665
228.000	-3.09E-04	-3108.7392	-458.9849	8.671E-06	168.2198	24.1206
234.000	-2.58E-04	-5432.3076	-323.8890	8.235E-06	169.9319	20.9113
240.000	-2.10E-04	-7002.8189	-208.1291	7.599E-06	171.0891	17.6753
246.000	-1.67E-04	-7936.6961	-111.5237	6.836E-06	171.7772	14.5265
252.000	-1.28E-04	-8347.2555	-33.2796	6.004E-06	172.0797	11.5549
258.000	-9.49E-05	-8341.4551	27.8685	5.151E-06	172.0754	8.8278
264.000	-6.66E-05	-8017.4698	73.5248	4.315E-06	171.8367	6.3910
270.000	-4.31E-05	-7463.0410	105.5099	3.524E-06	171.4282	4.2707
276.000	-2.43E-05	-6754.5233	125.7519	2.798E-06	170.9061	2.4766
282.000	-9.56E-06	-5956.5367	136.1945	2.148E-06	170.3182	1.0043
288.000	1.50E-06	-5122.1229	138.7214	1.582E-06	169.7033	-.1619
294.000	9.42E-06	-4293.3038	135.0978	1.101E-06	169.0926	-1.0459
300.000	1.47E-05	-3501.9401	126.9282	7.028E-07	168.5095	-1.6773
306.000	1.79E-05	-2770.7984	115.6286	3.823E-07	167.9708	-2.0892
312.000	1.93E-05	-2114.7408	102.4127	1.327E-07	167.4874	-2.3161
318.000	1.94E-05	-1541.9654	88.2880	-5.419E-08	167.0654	-2.3922
324.000	1.87E-05	-1055.2366	74.0616	-1.869E-07	166.7067	-2.3500
330.000	1.72E-05	-653.0580	60.3531	-2.742E-07	166.4104	-2.2195
336.000	1.54E-05	-330.7520	47.6119	-3.245E-07	166.1729	-2.0275
342.000	1.33E-05	-81.4226	36.1379	-3.455E-07	165.9892	-1.7971
348.000	1.12E-05	103.2141	26.1040	-3.444E-07	166.0053	-1.5475
354.000	9.18E-06	232.1352	17.5787	-3.273E-07	166.1002	-1.2943

360.000	7.29E-06	314.4524	10.5481	-2.993E-07	166.1609	-1.0493
366.000	5.59E-06	358.9813	4.9364	-2.649E-07	166.1937	-.8213
372.000	4.11E-06	373.9273	.6242	-2.275E-07	166.2047	-.6161
378.000	2.86E-06	366.6760	-2.5356	-1.896E-07	166.1994	-.4371
384.000	1.83E-06	343.6705	-4.7043	-1.533E-07	166.1824	-.2858
390.000	1.02E-06	310.3621	-6.0467	-1.199E-07	166.1579	-.1617
396.000	3.93E-07	271.2178	-6.7227	-9.021E-08	166.1290	-.063611
402.000	-6.54E-08	229.7711	-6.8811	-6.461E-08	166.0985	.010796
408.000	-3.83E-07	188.7025	-6.6559	-4.322E-08	166.0682	.064282
414.000	-5.84E-07	149.9393	-6.1634	-2.592E-08	166.0397	.099885
420.000	-6.94E-07	114.7651	-5.5016	-1.239E-08	166.0138	.1207
426.000	-7.33E-07	83.9308	-4.7504	-2.242E-09	165.9910	.1297
432.000	-7.21E-07	57.7625	-3.9721	4.998E-09	165.9718	.1297
438.000	-6.73E-07	36.2608	-3.2136	9.803E-09	165.9559	.1231
444.000	-6.03E-07	19.1903	-2.5078	1.264E-08	165.9433	.1121
450.000	-5.21E-07	6.1563	-1.8758	1.393E-08	165.9337	.098514
456.000	-4.36E-07	-3.3315	-1.3292	1.408E-08	165.9317	.083667
462.000	-3.52E-07	-9.8072	-.8721	1.340E-08	165.9364	.068705
468.000	-2.75E-07	-13.8090	-.5027	1.220E-08	165.9394	.054433
474.000	-2.06E-07	-15.8506	-.2152	1.068E-08	165.9409	.041398
480.000	-1.47E-07	-16.4012	-.001225	9.034E-09	165.9413	.029932
486.000	-9.75E-08	-15.8734	.1492	7.385E-09	165.9409	.020193
492.000	-5.81E-08	-14.6180	.2463	5.827E-09	165.9400	.012202
498.000	-2.76E-08	-12.9227	.3006	4.420E-09	165.9387	.005884
504.000	-5.07E-09	-11.0149	.3215	3.197E-09	165.9373	.001095
510.000	1.07E-08	-9.0671	.3178	2.170E-09	165.9359	-.002350
516.000	2.10E-08	-7.2036	.2967	1.339E-09	165.9345	-.004656
522.000	2.68E-08	-5.5073	.2647	6.895E-10	165.9333	-.006030
528.000	2.92E-08	-4.0279	.2266	2.023E-10	165.9322	-.006669
534.000	2.92E-08	-2.7884	.1863	-1.460E-10	165.9313	-.006751
540.000	2.75E-08	-1.7918	.1468	-3.801E-10	165.9305	-.006434
546.000	2.47E-08	-1.0267	.1099	-5.241E-10	165.9300	-.005846
552.000	2.12E-08	-.4720	.077133	-6.007E-10	165.9296	-.005090
558.000	1.75E-08	-.1006	.049138	-6.299E-10	165.9293	-.004242
564.000	1.36E-08	.1182	.026340	-6.290E-10	165.9293	-.003357
570.000	9.91E-09	.2161	.008866	-6.120E-10	165.9294	-.002467
576.000	6.30E-09	.2251	-.003302	-5.894E-10	165.9294	-.001589
582.000	2.84E-09	.1770	-.010237	-5.689E-10	165.9293	-7.233E-04
588.000	-5.23E-10	.1028	-.012002	-5.546E-10	165.9293	1.348E-04
594.000	-3.82E-09	.033464	-.008608	-5.476E-10	165.9292	9.966E-04
600.000	-7.09E-09	0.0000	0.0000	-5.459E-10	165.9292	.001873

Output Verification:

Computed forces and moments are within specified convergence limits.

Output Summary for Load Case No. 1:

```
Pile-head deflection              =      .03663200 in
Computed slope at pile head       =    -.00036276
Maximum bending moment            =    187235.909 lbs-in
Maximum shear force               =     -2371.097 lbs
Depth of maximum bending moment   =       108.000 in
Depth of maximum shear force      =       156.000 in
Number of iterations              =             5
Number of zero deflection points  =             5
```

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 2

Pile-head boundary conditions are Shear and Moment (BC Type 1)

```
Specified shear force at pile head    =      5000.000 lbs
Specified bending moment at pile head =          .000 in-lbs
Specified axial load at pile head     =     75000.000 lbs
```

(Zero moment for this load indicates free-head conditions)

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Soil Res p lbs/in
0.000	.091580	-6.790E-08	5000.0000	-9.069E-04	165.9292	0.0000
6.000	.086139	30408.1043	5000.0000	-9.053E-04	188.3348	0.0000
12.000	.080716	60814.8102	5000.0000	-9.007E-04	210.7393	0.0000
18.000	.075330	91218.7193	5000.0000	-8.929E-04	233.1418	0.0000
24.000	.070001	121618.4335	5000.0000	-8.820E-04	255.5412	0.0000
30.000	.064746	152012.5547	5000.0000	-8.681E-04	277.9365	0.0000
36.000	.059584	182399.6851	5000.0000	-8.510E-04	300.3266	0.0000
42.000	.054534	212778.4274	5000.0000	-8.308E-04	322.7106	0.0000
48.000	.049615	243147.3844	5000.0000	-8.075E-04	345.0873	0.0000
54.000	.044845	273505.1596	5000.0000	-7.811E-04	367.4558	0.0000
60.000	.040242	303850.3568	5000.0000	-7.516E-04	389.8150	0.0000
66.000	.035826	334181.5806	5000.0000	-7.190E-04	412.1639	0.0000
72.000	.031614	364497.4360	5000.0000	-6.833E-04	434.5015	0.0000
78.000	.027626	394796.5290	4751.3634	-6.445E-04	456.8268	-82.8789
84.000	.023880	422093.8270	4072.8785	-6.027E-04	476.9403	-143.2828
90.000	.020393	444213.5344	3092.4072	-5.585E-04	493.2387	-183.5410
96.000	.017179	459705.3360	1923.3466	-5.123E-04	504.6535	-206.1458
102.000	.014246	467754.7477	663.8363	-4.649E-04	510.5846	-213.6910
108.000	.011600	468089.7736	-603.6434	-4.171E-04	510.8314	-208.8023
114.000	.009241	460886.3913	-1812.2463	-3.696E-04	505.5237	-194.0654
120.000	.007165	446675.4609	-2910.3146	-3.232E-04	495.0527	-171.9574
126.000	.005362	426253.5222	-3860.5465	-2.786E-04	480.0052	-144.7866
132.000	.003821	400599.6644	-4638.8328	-2.364E-04	461.1027	-114.6422
138.000	.002526	370800.2652	-5232.8317	-1.970E-04	439.1457	-83.3575
144.000	.001458	337982.9450	-5640.3601	-1.607E-04	414.9649	-52.4853
150.000	5.97E-04	303260.6099	-5867.6779	-1.280E-04	389.3805	-23.2873
156.000	-7.78E-05	267685.9867	-5927.7425	-9.880E-05	363.1680	3.2657
162.000	-5.88E-04	232216.6191	-5838.5002	-7.326E-05	337.0332	26.4817
168.000	-9.57E-04	197689.9141	-5621.2729	-5.129E-05	311.5929	45.9274
174.000	-.001204	164807.5041	-5299.2877	-3.277E-05	287.3642	61.4010
180.000	-.001350	134127.9509	-4896.3837	-1.749E-05	264.7586	72.9003
186.000	-.001414	106066.6409	-4435.9183	-5.217E-06	244.0822	80.5881
192.000	-.001413	80901.6264	-3939.8839	4.337E-06	225.5399	84.7567
198.000	-.001362	58784.1315	-3428.2358	1.147E-05	209.2431	85.7927
204.000	-.001275	39752.4696	-2918.4234	1.651E-05	195.2200	84.1448
210.000	-.001164	23748.1922	-2425.1076	1.975E-05	183.4276	80.2938
216.000	-.001038	10633.3999	-1960.0453	2.151E-05	173.7642	74.7269
222.000	-9.06E-04	208.2886	-1532.1158	2.206E-05	166.0827	67.9162
228.000	-7.73E-04	-7771.8479	-1147.4623	2.168E-05	171.6557	60.3016
234.000	-6.45E-04	-13580.7690	-809.7226	2.059E-05	175.9359	52.2783
240.000	-5.26E-04	-17507.0472	-520.3227	1.900E-05	178.8289	44.1883
246.000	-4.17E-04	-19841.7402	-278.8092	1.709E-05	180.5492	36.3162
252.000	-3.21E-04	-20868.1388	-83.1990	1.501E-05	181.3055	28.8872
258.000	-2.37E-04	-20853.6378	69.6712	1.288E-05	181.2948	22.0695
264.000	-1.66E-04	-20043.6745	183.8121	1.079E-05	180.6980	15.9774
270.000	-1.08E-04	-18657.6024	263.7747	8.811E-06	179.6767	10.6768
276.000	-6.07E-05	-16886.3082	314.3797	6.995E-06	178.3715	6.1915
282.000	-2.39E-05	-14891.3416	340.4862	5.371E-06	176.9016	2.5106
288.000	3.75E-06	-12805.3074	346.8035	3.956E-06	175.3645	-.4049
294.000	2.36E-05	-10733.2596	337.7446	2.753E-06	173.8378	-2.6148
300.000	3.68E-05	-8754.8503	317.3204	1.757E-06	172.3800	-4.1933
306.000	4.46E-05	-6926.9960	289.0715	9.557E-07	171.0332	-5.2230
312.000	4.83E-05	-5286.8520	256.0317	3.316E-07	169.8247	-5.7902
318.000	4.86E-05	-3854.9135	220.7199	-1.355E-07	168.7696	-5.9804
324.000	4.66E-05	-2638.0914	185.1540	-4.673E-07	167.8730	-5.8749

330.000	4.30E-05	-1632.6449	150.8829	-6.855E-07	167.1322	-5.5488
336.000	3.84E-05	-826.8801	119.0299	-8.112E-07	166.5385	-5.0689
342.000	3.33E-05	-203.5565	90.3448	-8.638E-07	166.0792	-4.4928
348.000	2.80E-05	258.0352	65.2600	-8.610E-07	166.1193	-3.8688
354.000	2.29E-05	580.3380	43.9466	-8.182E-07	166.3568	-3.2356
360.000	1.82E-05	786.1311	26.3701	-7.484E-07	166.5084	-2.6232
366.000	1.40E-05	897.4532	12.3409	-6.623E-07	166.5905	-2.0532
372.000	1.03E-05	934.8182	1.5604	-5.687E-07	166.6180	-1.5403
378.000	7.14E-06	916.6899	-6.3391	-4.741E-07	166.6046	-1.0929
384.000	4.58E-06	859.1761	-11.7608	-3.834E-07	166.5623	-.7144
390.000	2.54E-06	775.9051	-15.1168	-2.998E-07	166.5009	-.4043
396.000	9.82E-07	678.0445	-16.8067	-2.255E-07	166.4288	-.1590
402.000	-1.64E-07	574.4277	-17.2028	-1.615E-07	166.3525	.026990
408.000	-9.57E-07	471.7562	-16.6397	-1.081E-07	166.2768	.1607
414.000	-1.46E-06	374.8483	-15.4085	-6.480E-08	166.2054	.2497
420.000	-1.73E-06	286.9128	-13.7541	-3.099E-08	166.1406	.3017
426.000	-1.83E-06	209.8271	-11.8760	-5.604E-09	166.0838	.3243
432.000	-1.80E-06	144.4062	-9.9303	1.250E-08	166.0356	.3243
438.000	-1.68E-06	90.6519	-8.0340	2.451E-08	165.9960	.3078
444.000	-1.51E-06	47.9758	-6.2694	3.159E-08	165.9646	.2804
450.000	-1.30E-06	15.3907	-4.6894	3.483E-08	165.9405	.2463
456.000	-1.09E-06	-8.3288	-3.3231	3.519E-08	165.9353	.2092
462.000	-8.81E-07	-24.5179	-2.1803	3.351E-08	165.9473	.1718
468.000	-6.87E-07	-34.5225	-1.2568	3.049E-08	165.9546	.1361
474.000	-5.15E-07	-39.6265	-.5380	2.671E-08	165.9584	.1035
480.000	-3.67E-07	-41.0030	-.003061	2.259E-08	165.9594	.074830
486.000	-2.44E-07	-39.6836	.3729	1.846E-08	165.9584	.050482
492.000	-1.45E-07	-36.5451	.6158	1.457E-08	165.9561	.030506
498.000	-6.91E-08	-32.3066	.7515	1.105E-08	165.9530	.014711
504.000	-1.27E-08	-27.5371	.8038	7.991E-09	165.9495	.002738
510.000	2.68E-08	-22.6678	.7944	5.426E-09	165.9459	-.005876
516.000	5.24E-08	-18.0090	.7419	3.347E-09	165.9425	-.011641
522.000	6.70E-08	-13.7684	.6617	1.724E-09	165.9393	-.015075
528.000	7.31E-08	-10.0698	.5665	5.056E-10	165.9366	-.016671
534.000	7.31E-08	-6.9710	.4658	-3.651E-10	165.9343	-.016878
540.000	6.87E-08	-4.4794	.3669	-9.502E-10	165.9325	-.016085
546.000	6.17E-08	-2.5667	.2748	-1.310E-09	165.9311	-.014614
552.000	5.30E-08	-1.1801	.1928	-1.502E-09	165.9301	-.012724
558.000	4.36E-08	-.2514	.1228	-1.575E-09	165.9294	-.010606
564.000	3.41E-08	.2955	.065850	-1.573E-09	165.9294	-.008393
570.000	2.48E-08	.5402	.022165	-1.530E-09	165.9296	-.006168
576.000	1.58E-08	.5628	-.008255	-1.474E-09	165.9296	-.003971
582.000	7.09E-09	.4425	-.025593	-1.422E-09	165.9295	-.001808
588.000	-1.31E-09	.2570	-.030006	-1.386E-09	165.9294	3.371E-04
594.000	-9.55E-09	.083660	-.021520	-1.369E-09	165.9293	.002492
600.000	-1.77E-08	0.0000	0.0000	-1.365E-09	165.9292	.004682

Output Verification:

Computed forces and moments are within specified convergence limits.

Output Summary for Load Case No. 2:

```
Pile-head deflection            =      .09158001 in
Computed slope at pile head     =     -.00090690
Maximum bending moment          =     468089.774 lbs-in
Maximum shear force             =      -5927.743 lbs
Depth of maximum bending moment =        108.000 in
Depth of maximum shear force    =        156.000 in
Number of iterations            =              5
Number of zero deflection points =             5
```


Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 3

```
Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head    =      10000.000 lbs
Specified bending moment at pile head =           .000 in-lbs
Specified axial load at pile head     =      75000.000 lbs
```

(Zero moment for this load indicates free-head conditions)

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Soil Res p lbs/in
0.000	.189083	2.716E-07	10000.0000	-.001854	165.9292	0.0000
6.000	.177957	60834.4855	10000.0000	-.001851	210.7538	0.0000
12.000	.166868	121666.1734	10000.0000	-.001842	255.5764	0.0000
18.000	.155853	182492.2661	10000.0000	-.001826	300.3948	0.0000
24.000	.144950	243309.9664	10000.0000	-.001805	345.2071	0.0000
30.000	.134197	304116.4774	10000.0000	-.001777	390.0111	0.0000
36.000	.123630	364909.0026	10000.0000	-.001743	434.8048	0.0000
42.000	.113287	425684.7464	10000.0000	-.001702	479.5861	0.0000
48.000	.103204	486440.9139	10000.0000	-.001656	524.3531	0.0000
54.000	.093420	547174.7109	10000.0000	-.001603	569.1035	0.0000
60.000	.083972	607883.3445	10000.0000	-.001544	613.8354	0.0000
66.000	.074896	668564.0228	10000.0000	-.001478	658.5467	0.0000
72.000	.066231	729213.9552	10000.0000	-.001407	703.2353	0.0000
78.000	.058012	789830.3525	9477.8925	-.001329	747.8993	-174.0358
84.000	.050278	844145.1377	8269.9576	-.001246	787.9200	-228.6092
90.000	.043061	890191.1729	6725.3975	-.001157	821.8480	-286.2442
96.000	.036390	925891.4782	4753.1784	-.001065	848.1531	-371.1621
102.000	.030287	948187.3664	2276.7834	-9.687E-04	864.5813	-454.3029
108.000	.024765	954084.7462	-423.4412	-8.715E-04	868.9267	-445.7720
114.000	.019828	943890.4578	-3009.9452	-7.746E-04	861.4152	-416.3960
120.000	.015470	918662.5049	-5373.0037	-6.794E-04	842.8265	-371.2901
126.000	.011676	880025.8603	-7432.6106	-5.875E-04	814.3579	-315.2455
132.000	.008421	829999.9067	-9136.2106	-5.001E-04	777.4974	-252.6212
138.000	.005675	770841.4208	-10455.8581	-4.183E-04	733.9077	-187.2613
144.000	.003401	704906.0785	-11384.9633	-3.429E-04	685.3246	-122.4404
150.000	.001560	634530.4635	-11934.7918	-2.744E-04	633.4698	-60.8357
156.000	1.08E-04	561935.5815	-12130.8735	-2.133E-04	579.9797	-4.5248
162.000	-10.0E-03	489151.9627	-12009.4672	-1.596E-04	526.3506	44.9936
168.000	-.001808	417965.6187	-11614.2041	-1.133E-04	473.8985	86.7608
174.000	-.002359	349883.4404	-10993.0120	-7.402E-05	423.7335	120.3033
180.000	-.002696	286116.0907	-10195.3951	-4.152E-05	376.7479	145.5690
186.000	-.002857	227576.0667	-9270.1210	-1.527E-05	333.6139	162.8557
192.000	-.002879	174888.3816	-8263.3396	5.294E-06	294.7921	172.7381
198.000	-.002794	128411.2265	-7217.1379	2.079E-05	260.5463	175.9958
204.000	-.002629	88264.0142	-6168.5173	3.186E-05	230.9647	173.5444
210.000	-.002411	54360.3409	-5148.7624	3.915E-05	205.9835	166.3739
216.000	-.002160	26443.6285	-4183.1586	4.328E-05	185.4136	155.4940
222.000	-.001892	4123.4854	-3291.0106	4.484E-05	168.9675	141.8886
228.000	-.001622	-13088.8573	-2485.9071	4.438E-05	175.5735	126.4792
234.000	-.001359	-25747.3457	-1776.1754	4.240E-05	184.9006	110.0980
240.000	-.001113	-34441.1216	-1165.4738	3.932E-05	191.3064	93.4692
246.000	-8.87E-04	-39768.4230	-653.4704	3.553E-05	195.2317	77.1986
252.000	-6.86E-04	-42314.7452	-236.5634	3.134E-05	197.1079	61.7704
258.000	-5.11E-04	-42635.3882	91.3950	2.700E-05	197.3442	47.5491
264.000	-3.62E-04	-41242.3031	338.4039	2.271E-05	196.3177	34.7872
270.000	-2.39E-04	-38594.9821	513.6717	1.863E-05	194.3671	23.6354
276.000	-1.39E-04	-35095.0119	627.0450	1.487E-05	191.7882	14.1557
282.000	-6.03E-05	-31083.8222	688.5195	1.149E-05	188.8327	6.3358
288.000	-9.60E-07	-26843.1145	707.8379	8.525E-06	185.7080	.1037
294.000	4.20E-05	-22597.4398	694.1759	5.999E-06	182.5797	-4.6577

300.000	7.10E-05	-18518.4033	655.9116	3.898E-06	179.5741	-8.0970
306.000	8.87E-05	-14730.0084	600.4739	2.199E-06	176.7827	-10.3822
312.000	9.74E-05	-11314.6958	534.2578	8.682E-07	174.2662	-11.6898
318.000	9.92E-05	-8319.6960	462.6000	-1.351E-07	172.0594	-12.1961
324.000	9.58E-05	-5763.3740	389.8014	-8.547E-07	170.1758	-12.0701
330.000	8.89E-05	-3641.3105	319.1870	-1.335E-06	168.6122	-11.4680
336.000	7.98E-05	-1931.9285	253.1933	-1.620E-06	167.3527	-10.5299
342.000	6.95E-05	-601.5327	193.4727	-1.749E-06	166.3724	-9.3770
348.000	5.88E-05	391.3182	141.0075	-1.760E-06	166.2175	-8.1114
354.000	4.83E-05	1092.1412	96.2268	-1.684E-06	166.7339	-6.8155
360.000	3.86E-05	1547.5558	59.1200	-1.550E-06	167.0695	-5.5534
366.000	2.97E-05	1802.9763	29.3433	-1.378E-06	167.2577	-4.3722
372.000	2.20E-05	1900.9156	6.3152	-1.189E-06	167.3299	-3.3038
378.000	1.55E-05	1879.8292	-10.6989	-9.959E-07	167.3143	-2.3675
384.000	1.01E-05	1773.4252	-22.5167	-8.092E-07	167.2359	-1.5717
390.000	5.76E-06	1610.3574	-29.9812	-6.363E-07	167.1158	-.9164
396.000	2.44E-06	1414.2238	-33.9162	-4.817E-07	166.9712	-.3952
402.000	-1.71E-08	1203.7967	-35.0934	-3.480E-07	166.8162	.002827
408.000	-1.74E-06	993.4160	-34.2101	-2.357E-07	166.6612	.2916
414.000	-2.85E-06	793.4881	-31.8755	-1.444E-07	166.5139	.4866
420.000	-3.47E-06	611.0403	-28.6052	-7.262E-08	166.3794	.6035
426.000	-3.72E-06	450.2910	-24.8210	-1.839E-08	166.2610	.6579
432.000	-3.69E-06	313.2048	-20.8552	2.063E-08	166.1600	.6640
438.000	-3.47E-06	200.0097	-16.9584	4.685E-08	166.0766	.6349
444.000	-3.13E-06	109.6614	-13.3089	6.267E-08	166.0100	.5816
450.000	-2.72E-06	40.2459	-10.0235	7.033E-08	165.9589	.5136
456.000	-2.28E-06	-10.6833	-7.1678	7.184E-08	165.9371	.4383
462.000	-1.86E-06	-45.8326	-4.7676	6.896E-08	165.9630	.3617
468.000	-1.46E-06	-67.9569	-2.8179	6.314E-08	165.9793	.2882
474.000	-1.10E-06	-79.7039	-1.2916	5.560E-08	165.9879	.2206
480.000	-7.88E-07	-83.5064	-.1475	4.726E-08	165.9907	.1608
486.000	-5.30E-07	-81.5162	.6642	3.883E-08	165.9893	.1098
492.000	-3.22E-07	-75.5705	1.1966	3.080E-08	165.9849	.067692
498.000	-1.61E-07	-67.1843	1.5024	2.350E-08	165.9787	.034233
504.000	-4.03E-08	-57.5627	1.6312	1.713E-08	165.9716	.008703
510.000	4.48E-08	-47.6251	1.6279	1.176E-08	165.9643	-.009820
516.000	1.01E-07	-38.0389	1.5313	7.378E-09	165.9572	-.022371
522.000	1.33E-07	-29.2562	1.3741	3.939E-09	165.9508	-.030010
528.000	1.48E-07	-21.5526	1.1829	1.343E-09	165.9451	-.033754
534.000	1.49E-07	-15.0631	.9780	-5.279E-10	165.9403	-.034533
540.000	1.42E-07	-9.8161	.7749	-1.799E-09	165.9364	-.033160
546.000	1.28E-07	-5.7625	.5845	-2.595E-09	165.9334	-.030313
552.000	1.11E-07	-2.7998	.4140	-3.033E-09	165.9313	-.026536
558.000	9.15E-08	-.7923	.2676	-3.216E-09	165.9298	-.022237
564.000	7.20E-08	.4148	.1478	-3.236E-09	165.9295	-.017705
570.000	5.27E-08	.9844	.055342	-3.164E-09	165.9299	-.013118
576.000	3.40E-08	1.0817	-.009718	-3.058E-09	165.9300	-.008568
582.000	1.60E-08	.8705	-.047650	-2.959E-09	165.9298	-.004076
588.000	-1.50E-09	.5126	-.058714	-2.888E-09	165.9296	3.877E-04
594.000	-1.87E-08	.1686	-.042929	-2.853E-09	165.9293	.004874
600.000	-3.57E-08	0.0000	0.0000	-2.845E-09	165.9292	.009436

Output Verification:

Computed forces and moments are within specified convergence limits.

Output Summary for Load Case No. 3:

Pile-head deflection	=	.18908331 in
Computed slope at pile head	=	-.00185441
Maximum bending moment	=	954084.746 lbs-in
Maximum shear force	=	-12130.873 lbs
Depth of maximum bending moment	=	108.000 in
Depth of maximum shear force	=	156.000 in
Number of iterations	=	7

Number of zero deflection points = 5

Summary of Pile-head Response

Definition of symbols for pile-head boundary conditions:

y = pile-head displacment, in
M = pile-head moment, lbs-in
V = pile-head shear force, lbs
S = pile-head slope, radians
R = rotational stiffness of pile-head, in-lbs/rad

BC Type	Boundary Condition 1	Boundary Condition 2	Axial Load lbs	Pile Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
1	V= 2000.000	M= 0.000	75000.0000	.036632	187235.9094	-2371.0970
1	V= 5000.000	M= 0.000	75000.0000	.091580	468089.7736	-5927.7425
1	V= 10000.000	M= 0.000	75000.0000	.1891	954084.7462	-12130.8735

The analysis ended normally.

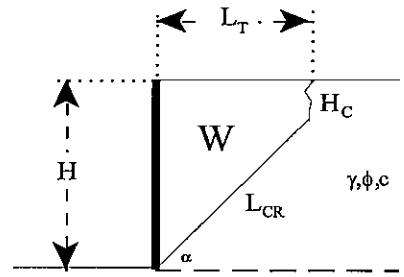
Geotechnologies, Inc.

Project: MGA North, LLC
File No.: 20572
Description: Retaining Walls

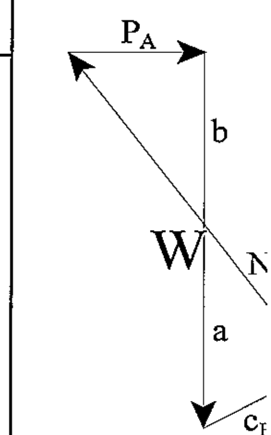
Retaining Wall Design with Level Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	15.00 feet
Unit Weight of Retained Soils	(γ)	125.0 pcf
Friction Angle of Retained Soils	(ϕ)	28.0 degrees
Cohesion of Retained Soils	(c)	150.0 psf
Factor of Safety	(FS)	1.50
Factored Parameters:	(ϕ_{FS})	19.5 degrees
	(c_{FS})	100.0 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	2.5	109	13678.5	17.7	3879.4	9799.1	4670.1
46	2.4	106	13222.4	17.5	3692.3	9530.0	4747.8
47	2.4	102	12778.9	17.2	3520.0	9259.0	4816.2
48	2.4	99	12347.7	17.0	3360.9	8986.8	4875.8
49	2.3	95	11928.0	16.8	3213.8	8714.3	4926.7
50	2.3	92	11519.4	16.6	3077.4	8442.0	4969.1
51	2.3	89	11121.2	16.3	2950.7	8170.4	5003.3
52	2.3	86	10732.9	16.1	2832.9	7900.0	5029.3
53	2.3	83	10353.9	15.9	2723.1	7630.9	5047.3
54	2.3	80	9983.9	15.7	2620.5	7363.4	5057.3
55	2.3	77	9622.2	15.5	2524.5	7097.7	5059.4
56	2.3	74	9268.4	15.4	2434.6	6833.9	5053.5
57	2.3	71	8922.2	15.2	2350.1	6572.1	5039.6
58	2.3	69	8583.0	15.0	2270.7	6312.3	5017.8
59	2.3	66	8250.5	14.8	2195.9	6054.6	4987.9
60	2.3	63	7924.3	14.6	2125.2	5799.1	4949.7
61	2.3	61	7604.0	14.5	2058.4	5545.6	4903.2
62	2.4	58	7289.2	14.3	1995.0	5294.2	4848.2
63	2.4	56	6979.7	14.1	1934.9	5044.8	4784.3
64	2.5	53	6675.0	14.0	1877.5	4797.5	4711.5
65	2.5	51	6375.0	13.8	1822.8	4552.1	4629.4
66	2.6	49	6079.2	13.6	1770.4	4308.7	4537.6
67	2.6	46	5787.3	13.5	1720.1	4067.2	4435.8
68	2.7	44	5499.1	13.3	1671.5	3827.6	4323.6
69	2.8	42	5214.3	13.1	1624.5	3589.8	4200.4
70	2.9	39	4932.5	12.9	1578.8	3353.8	4065.9



Design Equations (Vector Analysis):

$a = c_{FS} * L_{CR} * \sin(90-$

$b = W-a$

$P_A = b*\tan(\alpha-\phi_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\,max}$ 5059.4 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 45.0 pcf Use 45pcf

Geotechnologies, Inc.

Project: MGA North, LLC
File No.: 20572

Soil Weight	γ	125	pcf
Internal Friction Angle	ϕ	28	degrees
Cohesion	c	150	psf
Height of Retaining Wall	H	15	feet

Cantilever Retaining Wall Design based on At Rest Earth Pressure

$\sigma'_h = K_o\sigma'_v$

$K_o = 1 - \sin\phi$		0.531
$\sigma'_v = \gamma H$		1875.0 psf

$\sigma'_h =$ 994.7 psf

EFP = 66.3 pcf

$P_o =$ 7460.6 lbs/ft (based on a triangular distribution of pressure)

Design wall for an EFP of 66.3 pcf

Restrained Wall Design based on At Rest Earth Pressure

$P_o =$ 7460.6 lbs/ft

$\sigma'_{h,\,max} =$ 41.4 H (based on a trapezoidal distribution of pressure)

$\sigma'_{h,\,max} =$ 497.4 psf

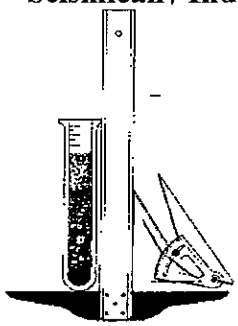
Design restrained wall for 41.4 H

Geotechnologies, Inc.

Project: MGA North, LLC

File No.: 20572

Seismically Induced Lateral Soil Pressure on Retaining Wall



Input:

Height of	(H)	15.0 feet
Retained	(γ)	125.0 pcf
Horizonta	(k_h)	0.24 g

Seismic I

ΔP_{AE} = (0

ΔP_{AE} = 2531.3 lbs/ft

Force applied at 0.6H above the base of the wall
Transfer load to 2/3 of the height of the wall

$T*(2/3)*H = \Delta P_{AE}*0.6*H$

T = 2278.1 lbs/ft

$EFP = 2*T/H^2$

EFP = 20.3 pcf

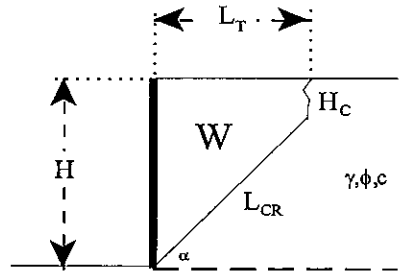
Geotechnologies, Inc.

Project: MGA North, LLC
File No.: 20572
Description: Shoring

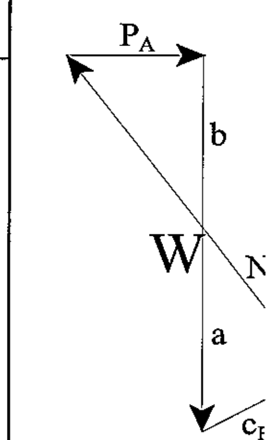
Shoring Design with Level Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	15.00 feet
Unit Weight of Retained Soils	(γ)	125.0 pcf
Friction Angle of Retained Soils	(ϕ)	28.0 degrees
Cohesion of Retained Soils	(c)	150.0 psf
Factor of Safety	(FS)	1.25
Factored Parameters:	(ϕ_{FS})	23.0 degrees
	(c_{FS})	120.0 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	3.3	107	13364.7	16.5	4869.4	8495.4	3424.9
46	3.3	104	12938.4	16.3	4620.4	8318.0	3523.3
47	3.2	100	12520.4	16.1	4391.5	8128.9	3611.8
48	3.1	97	12111.0	16.0	4180.6	7930.4	3690.7
49	3.1	94	11710.1	15.8	3985.9	7724.2	3760.1
50	3.0	91	11317.8	15.6	3805.8	7512.0	3820.4
51	3.0	87	10933.8	15.4	3638.8	7295.0	3871.7
52	3.0	84	10557.9	15.3	3483.8	7074.1	3914.3
53	2.9	82	10189.9	15.1	3339.5	6850.4	3948.2
54	2.9	79	9829.4	14.9	3204.9	6624.4	3973.5
55	2.9	76	9476.1	14.8	3079.3	6396.8	3990.5
56	2.9	73	9129.8	14.6	2961.6	6168.1	3999.0
57	2.9	70	8790.0	14.4	2851.3	5938.7	3999.2
58	2.9	68	8456.6	14.3	2747.7	5708.9	3991.0
59	2.9	65	8129.2	14.1	2650.1	5479.1	3974.4
60	2.9	62	7807.4	13.9	2558.0	5249.3	3949.4
61	3.0	60	7490.9	13.8	2470.9	5020.0	3915.9
62	3.0	57	7179.5	13.6	2388.4	4791.1	3873.8
63	3.0	55	6872.8	13.4	2310.0	4562.9	3822.8
64	3.1	53	6570.6	13.3	2235.2	4335.4	3762.9
65	3.1	50	6272.6	13.1	2163.8	4108.7	3693.9
66	3.2	48	5978.4	12.9	2095.4	3883.0	3615.5
67	3.3	46	5687.7	12.8	2029.5	3658.2	3527.4
68	3.3	43	5400.3	12.6	1965.8	3434.6	3429.4
69	3.4	41	5115.9	12.4	1904.0	3212.0	3321.1
70	3.5	39	4834.2	12.2	1843.6	2990.6	3202.2



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90\text{-}$

$b = W\text{-}a$

$P_A = b*\tan(\alpha-\phi_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 3999.2 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 35.5 pcf Use 36pcf

Geotechnologies, Inc.

Project: MGA North, LLC
File No.: 20572
Description: Typical Tieback Anchors at 52-68A

Pressure Grouted Tiebacks

(Ref: California Trenching and Shoring Manual. Rev. 12)

Input:

H	15.0 feet	*height of shoring*
h	8.5 feet	*depth to anchor from top of shoring*
a	35.0 degrees	*active wedge failiure angle*
B	20.0 degrees	*installation angle of anchor shaft*
p_i	300.0 psi	*injection pressure during grouting*
d_s	6.0 inches	*diameter of anchor (bonded zone)*
L_s	20.0 feet	*length of anchor shaft of bonded zone*
ϕ_e	26.0 degrees	*effective friction angle between soil and grout*

$P_u = p_i * \pi * d_s * L_s * \tan\phi_e$

661.9 kips	**ultimate capacity**
441.3 kips	**allowable capacity with 1.5 factor of safety**
14.0 ksf	**allowable skin friction**
14046.7 psf	**allowable skin friction**
98.0 kips	***actual design load by shoring consultant***
3.1194 ksf	***actual skin friction***
3119.4 psf	***actual skin friction***

Geotechnologies, Inc.

Tiebacks Calculations

(Ref: US Army Corps of Engineers, AMF 88-3)

Project: MGA North, LLC

File No. 20572

Location: Top Row of Tiebacks

Soil Parameters:

Weight of Soil	γ	125.00	*lbs/ft³*
Friction Angle	ϕ	28.00	*degrees*
Cohesion	c	150.00	*lbs/ft²*
Tieback Angle	α	20.00	*degrees*
Earth Pressure Coefficient	K	0.50	

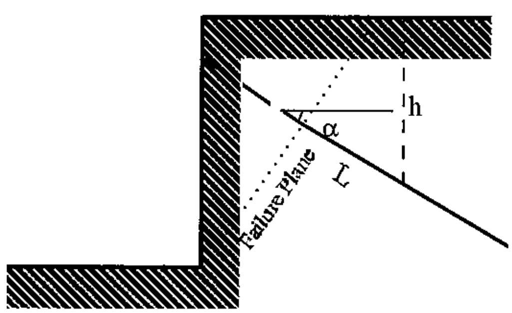
Design Assumptions:

Diameter of Grout	d	1.00	*feet*
Length of Embeddment	L	20.00	*feet*
Depth to midpoint of Embeddment	h	12.00	*feet*
Factor of Safety Applied	F.S.	1.50	
Normal Stress $(\sigma v+K\sigma v)/2+(\sigma v-K\sigma v)/2*\cos 2\alpha$	$\sigma n =$	1412.27	
Ultimate Resistance:	R_{ult}	56.61	*kips*

Eq: $pi*d*\gamma*L*\sigma n*\tan(\phi)+c*pi*d*L$

Allowable Resistance:	$R_{allow} = R_{ult}/F.S.$	37.74	*kips*
Allowable Skin Friction:	$R_{allow}/2/pi/1/L$	600.61	*psf*

Allowable Skin Friction Design Value **600** ***psf***



BORING LOG NUMBER 1

Drilling Date: 04/24/08

Project: File No. 19668 **MGA North, LLC**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: 8½-inch Concrete over 3½-inch Base
				0 --		FILL: Silty Sand, dark and yellowish-brown, moist, dense, fine grained
				-		
1	70	11.6	117.3	1 --		
				-		
				2 --		
				-		
3	75	12.9	119.4	3 --		
				-		
				4 --		
				-		
5	47	13.6	115.9	5 --		
				-		
				6 --		
				-		
7	48	11.2	118.2	7 --		
				-	ML	Sandy Silty, yellowish-brown, moist, stiff
				8 --		
				-		
				9 --		
				-		
10	70	11.3	120.7	10 --		
				-	SM/SC	Silty to Clayey Sand, yellowish-brown, moist, dense, fine grained, stiff
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	80	16.3	112.2	15 --		
				-	SC	Clayey Sand, yellowish-brown, moist, very dense, fine grained, very stiff
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	50	16.5	110.6	20 --		moist
				-		
				21 --		Total depth: 20 feet; No Water; Fill to 7 feet
				-		
				22 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual
				23 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				24 --		140-lb. Slide Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted
				25 --		
				-		SPT=Standard Penetration Test

BORING LOG NUMBER 2

Drilling Date: 04/24/08

Project: File No. 19668 MGA North, LLC

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: 8-inch Concrete over 3½-inch Base
				0 --		FILL: Silty Sand, yellowish-brown, moist, medium dense, fine grained
				-		
				1 --		
				-		
				2 --		
2.5	17	15.6	88.5	-	SP	Sand, yellowish-brown, moist, medium dense, fine grained
				3 --		
				-	SP/ML	Sand to Sandy Silt, yellowish-brown, moist, medium dense, fine grained
				4 --		
				-		
5	14	3.4	SPT	5 --		
				-	SP	Sand, yellowish-brown, moist, medium dense, fine to medium grained
				6 --		
				-		
				7 --		
7.5	14	13.2	93.9	-		
				8 --	SM	Silty Sand, yellowish-brown, moist, medium dense, fine grained
				-		
				9 --		
				-		
10	10	17.6	SPT	10 --		
				-	SM/SC	Silty to Clayey Sand, yellowish-brown, moist, medium dense, fine grained, firm
				11 --		
				-		
				12 --		
12.5	42	13.3	106.4	-		
				13 --		moist
				-		
				14 --		
				-		
15	12	17.9	SPT	15 --		
				-	SC/SM	Clayey to Silty Sand, yellowish-brown, moist, medium dense, fine grained, firm
				16 --		
				-		
				17 --		
17.5	25	17.0	101.7	-		
				18 --	SC/SM	Clayey to Silty Sand, yellowish-brown, slightly porous, moist, medium dense, fine grained, firm
				-		
				19 --		
				-		
20	20	14.7	SPT	20 --		
				-	SM	Silty Sand, yellowish-brown, moist, medium dense, fine grained
				21 --		
				-		
				22 --		
22.5	37	21.1	101.2	-		
				23 --	SC	Clayey Sand, yellowish-brown, moist, medium dense, fine grained, firm
				-		
				24 --		
				-		
25	18	23.2	SPT	25 --		Silty to Clayey Sand, yellowish-brown, moist, medium dense, fine grained, firm
				-	SM/SC	

BORING LOG NUMBER 2

Project: File No. 19668 **MGA North, LLC**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				26 --		
				-	SP	Sand, yellowish-brown, moist, medium dense, fine grained
				27 --		
27.5	32	24.6	95.6	-		
				28 --	CL	Silty to Sandy Clay, yellowish-brown, moist, firm
				-		
				29 --		
				-		
30	17	15.0	SPT	30 --		
				-	SC/SM	Clayey to Silty Sand, yellowish-brown, moist, medium dense, fine grained, firm, slight gravel
				31 --		
				-		
				32 --		
32.5	25 50/6"	11.1	115.5	-		
				33 --	SM	Silty Sand with slight Clay, yellowish-brown, moist, dense, fine grained
				-		
				34 --		
				-		
35	35	14.1	SPT	35 --		
				-	SM/SC	Silty to Clayey Sand, yellowish-brown, moist, medium dense, fine grained, firm
				36 --		
				-		
				37 --		
37.5	36	14.1	106.5	-		
				38 --		slightly porous, moist
				-		
				39 --		
				-		
40	30	14.0	SPT	40 --		
				-	SC	Clayey Sand, yellowish-brown, moist, medium dense, fine grained, firm
				41 --		
				-		
				42 --		
42.5	75/7"	21.1	105.3	-		
				43 --	SC/CL	Clayey Sand to Sandy Clay, yellowish-brown, moist, dense, fine grained, stiff
				-		
				44 --		
				-		
45	26	17.8	SPT	45 --		
				-	SC/SM	Clayey to Silty Sand, yellowish-brown, moist, medium dense, fine grained, firm, slight gravel
				46 --		
				-		
				47 --		
47.5	78	9.3	97.8	-		
				48 --	SW	Sand with Gravel, yellowish-brown, moist, dense, fine grained
				-		
				49 --		
				-		
50	50/6"	8.4	SPT	50 --		moist, dense, fine to medium grained
				-		Total depth: 50 feet; No Water; Fill to 2 feet

BORING LOG NUMBER 3

Drilling Date: 04/24/08

Project: File No. 19668 **MGA North, LLC**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: 8-inch Concrete over 3½-inch Base
				0 --		FILL: Silty Sand, yellowish-brown, moist, medium dense, fine grained
				-		
1	37	17.0	106.1	1 --		
				-		gray with yellowish-brown mottling, moist, medium dense, fine grained
				2 --		
				-		
3	51	20.4	106.8	3 --		
				-		Clayey to Silty Sand, gray to yellowish-brown, moist, medium dense, fine grained, firm, slight gravel
				4 --		
				-		
5	29	16.7	107.9	5 --		
				-	SC	Clayey Sand, yellowish-brown, slightly porous, moist, medium dense, fine grained, firm
				6 --		
				-		
7	14	16.0	97.0	7 --		
				-	SM	Silty Sand, yellowish-brown, slightly porous, slight caliche, moist, dense, fine grained
				8 --		
				-		
				9 --		
				-		
10	15	14.4	100.6	10 --		
				-	SM/SC	Silty to Clayey Sand, yellowish-brown, slight caliche, slightly porous, moist, medium dense, fine grained, firm
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	41	16.4	101.5	15 --		
				-	SW/SC	Sand with Gravel to Clayey Sand, yellowish-brown, moist, medium dense, fine to medium grained
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	25	23.7	94.5	20 --	SC/CL	Clayey Sand to Silty Clay, yellowish-brown, moist, medium dense, fine grained, firm
				-		
				21 --		
				-		Total depth: 20 feet No Water Fill to 5 feet
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
				25 --		
				-		

BORING LOG NUMBER 4

Drilling Date: 04/24/08

Project: File No. 19668 **MGA North, LLC**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Lawn Area
				0 --		FILL: Silty Sand, yellowish-brown, moist, medium dense, fine grained
				1 --		
2	19	9.3	105.8	2 --	SM	Silty Sand, yellowish-brown, slightly porous, moist, medium dense, fine grained
				3 --		moist
4	22	10.3	101.7	4 --		slightly porous, slight caliche, moist
				5 --		
				6 --		
7	16	16.0	98.8	7 --		slightly Clayey, yellowish-brown, slightly porous, slight caliche, moist
				8 --		
				9 --		
10	20	23.2	96.2	10 --	SM/SC	Silty to Clayey Sand, yellowish-brown, slightly porous, moist, medium dense, fine grained
				11 --		
				12 --		
				13 --		
				14 --		
15	29	23.1	92.7	15 --	SC/CL	Clayey Sand to Sandy Clay, yellowish-brown, moist, medium dense, fine grained, firm
				16 --		
				17 --		
				18 --		
				19 --		
20	35	23.5	93.3	20 --	CL	Sandy Clay, yellowish-brown, moist, firm
				21 --		Total depth: 20 feet No Water Fill to 1½ feet
				22 --		
				23 --		
				24 --		
				25 --		

BORING LOG NUMBER 5

Drilling Date: 04/24/08

Project: File No. 19668 **MGA North, LLC**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: 8-inch Concrete over 3-inch Base
				0 --		FILL: Silty Sand, yellowish-brown, moist, medium dense, fine grained
1	16	13.0	109.9	1 --	SM	Silty Sand, yellowish-brown, moist, medium dense, fine grained
				2 --		
3	15	11.9	91.8	3 --		moist
				4 --		
5	11	14.7	99.4	5 --	SM/SC	Silty to Clayey Sand, yellowish-brown, moist, medium dense, fine grained, firm
				6 --		
7	17	15.0	95.7	7 --		moist
				8 --		
				9 --		
10	23	23.5	95.2	10 --	SC	Clayey Sand, yellowish-brown, moist, medium dense, fine grained, firm
				11 --		
				12 --		
				13 --		
				14 --		
15	39	17.9	102.1	15 --	SC/CL	Clayey Sand to Sandy Clay, yellowish-brown, moist, medium dense, fine grained, firm
				16 --		
				17 --		
				18 --		
				19 --		
20	47	6.9	102.4	20 --	SM	Silty Sand, yellowish-brown, moist, medium dense, fine grained
				21 --		Total depth: 20 feet No Water Fill to 1 foot
				22 --		
				23 --		
				24 --		
				25 --		

LOG OF TEST PIT NUMBER 1

Drilling Date: 04/25/08

Project: File No. 19668 **MGA North, LLC**

km

Sample Depth ft.	Moisture Content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Weeds
			0 -- -		FILL: Silty Sand, yellowish-brown, slightly moist, medium dense, fine grained
			1 -- -		
			2 -- -		
			3 -- -	SM	Silty Sand, yellowish-brown, slightly moist, medium dense, fine grained
			4 -- -		
			5 -- -		
			6 -- -		Total depth: 6 feet No Water Fill to 3 feet
			7 -- -		
			8 -- -		
			9 -- -		
			10 -- -		
			11 -- -		
			12 -- -		
			13 -- -		
			14 -- -		
			15 -- -		
			16 -- -		
			17 -- -		
			18 -- -		
			19 -- -		
			20 -- -		
			21 -- -		
			22 -- -		
			23 -- -		
			24 -- -		
			25 -- -		

LOG OF TEST PIT NUMBER 2

Drilling Date: 04/25/08

Project: File No. 19668 **MGA North, LLC**

km

Sample Depth ft.	Moisture Content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Ivy
			0 --		FILL: Silty Sand, dark and yellowish-brown, moist, medium dense, minor gravel
1	6.0	122.4	1 --		
			2 --	SM	Silty Sand, yellowish-brown, slightly moist, medium dense, fine grained
3	4.5	108.6	3 --		
			4 --		
5	7.7	103.0	5 --	ML	Sandy Silt, yellowish-brown, slightly moist, stiff, minor caliche
			6 --		
7	7.6	96.7	7 --	SM	Silty Sand, yellowish-brown, slightly moist, medium dense, fine grained
			8 --		
			9 --		
10	4.8	102.3	10 --	SM/SP	Silty Sand to Sand, yellowish-brown, slightly moist, medium dense, fine grained
			11 --		
			12 --		
			13 --		
			14 --		
15	8.4	92.8	15 --	SM	Silty Sand, yellowish-brown, slightly moist, medium dense, fine grained
			16 --		
			17 --		
			18 --		
			19 --		
20	7.6	111.8	20 --		Silty Sand, yellowish-brown, moist, medium dense to dense, fine grained
			21 --		Total depth: 20 feet No Water Fill to 2 feet
			22 --		
			23 --		
			24 --		
			25 --		

August 13, 2013 via email: Ehill@geoteq.com

GEOTECHNOLOGIES, INC.
439 Western Ave
Glendale, CA 91201

Attention: Mr. Ed Hill

Re: Soil Corrosivity Study
MGA North LLC
Los Angeles, CA
HDR #215695

INTRODUCTION

Laboratory tests have been completed on one soil sample provided by Geotechnologies, Inc. for the MGA North LLC project. The purpose of these tests was to determine if the soil might have deleterious effects on underground utility piping, hydraulic elevator cylinders, and concrete structures. HDR Engineering, Inc. (HDR|Schiff) assumes that the sample provided is representative of the most corrosive soils at the site.

The proposed construction consists of a 3 story building. The site is located in Chatsworth, California. The water table is reportedly 50 feet deep.

The scope of this study is limited to a determination of soil corrosivity and general corrosion control recommendations for materials likely to be used for construction. Our recommendations do not constitute, and are not meant as a substitute for, design documents for the purpose of construction. If the architects and/or engineers desire more specific information, designs, specifications, or review of design, HDR|Schiff will be happy to work with them as a separate phase of this project.

LABORATORY SOIL CORROSIVITY TESTS

The electrical resistivity of the sample was measured in a soil box per ASTM G187 in its as-received condition and again after saturation with distilled water. Resistivities are at about their lowest value when the soil is saturated. The pH of the saturated sample was measured per CTM 643. A 5:1 water:soil extract from each sample was chemically analyzed for the major soluble salts commonly found in soil per ASTM D4327 and D6919. Laboratory analysis was performed under HDR|Schiff number 13-0572SCS and the test results are shown in Table 1.

SOIL CORROSIVITY

A major factor in determining soil corrosivity is electrical resistivity. The electrical resistivity of a soil is a measure of its resistance to the flow of electrical current. Corrosion of buried metal is an electrochemical process in which the amount of metal loss due to corrosion is directly proportional to the flow of electrical current (DC) from the metal into the soil. Corrosion currents, following Ohm's Law, are inversely proportional to soil resistivity. Lower electrical resistivities result from higher moisture and soluble salt contents and indicate corrosive soil.

A correlation between electrical resistivity and corrosivity toward ferrous metals is:[1]

Soil Resistivity in ohm-centimeters	Corrosivity Category
Greater than 10,000	Mildly Corrosive
2,000 to 10,000	Moderately Corrosive
1,000 to 2,000	Corrosive
0 to 1,000	Severely Corrosive

Other soil characteristics that may influence corrosivity towards metals are pH, soluble salt content, soil types, aeration, anaerobic conditions, and site drainage.

Electrical resistivity was in the mildly corrosive category with as-received moisture. When saturated, the resistivity was in the moderately corrosive category. The resistivity dropped considerably with added moisture because the sample was dry as-received.

Soil pH value was 7.5. This range is mildly alkaline.[2] This value does not particularly increase soil corrosivity.

The soluble salt content of the sample was low.

Tests were not made for sulfide and negative oxidation-reduction (redox) potential because this sample did not exhibit characteristics typically associated with anaerobic conditions.

This soil is classified as moderately corrosive to ferrous metals.

CORROSION CONTROL RECOMMENDATIONS

The life of buried materials depends on thickness, strength, loads, construction details, soil moisture, etc., in addition to soil corrosivity, and is, therefore, difficult to predict. Of more practical value are

[1] *Romanoff, Melvin. Underground Corrosion, NBS Circular 579. Reprinted by NACE. Houston, TX, 1989, pp. 166–167.*
[2] *Romanoff, Melvin. Underground Corrosion, NBS Circular 579. Reprinted by NACE. Houston, TX, 1989, p. 8.*

corrosion control methods that will increase the life of materials that would be subject to significant corrosion.

The following recommendations are based on the soil conditions discussed in the Soil Corrosivity section above. Unless otherwise indicated, these recommendations apply to the entire site or alignment.

Steel Pipe

Implement *all* the following measures:

1. Underground steel pipe with rubber gasketed, mechanical, grooved end, or other nonconductive type joints should be bonded for electrical continuity. Electrical continuity is necessary for corrosion monitoring and cathodic protection.
2. Install corrosion monitoring test stations to facilitate corrosion monitoring and the possible future application of cathodic protection:
    a. At each end of the pipeline.
    b. At each end of all casings.
    c. Other locations as necessary so the interval between test stations does not exceed 1,200 feet.
3. To prevent dissimilar metal corrosion cells and to facilitate the possible future application of cathodic protection, electrically isolate each buried steel pipeline per NACE Standard SP0286 from:
    a. Dissimilar metals.
    b. Dissimilarly coated piping (cement-mortar vs. dielectric).
    c. Above ground steel pipe.
    d. All existing piping.
4. Choose one of the following corrosion control options:

    OPTION 1

    a. Apply a suitable dielectric coating intended for underground use such as:
        i. Polyurethane per AWWA C222 *or*
        ii. Extruded polyethylene per AWWA C215 *or*
        iii. A tape coating system per AWWA C214 *or*
        iv. Hot applied coal tar enamel per AWWA C203 *or*
        v. Fusion bonded epoxy per AWWA C213.
    b. Although it is customary to cathodically protect bonded dielectrically coated structures, cathodic protection is not recommended at this time due to moderately corrosive soils. Joint bonds, test stations, and insulated joints should still be installed and will facilitate the application of cathodic protection in the future if needed to control leaks.

OPTION 2

a. As an alternative to dielectric coating and cathodic protection, apply a ¾-inch cement mortar coating per AWWA C205 or encase in concrete 3 inches thick, using any type of cement. Joint bonds, test stations, and insulated joints are still required for these alternatives.

NOTE: Some steel piping systems, such as for oil, gas, and high-pressure piping systems, have special corrosion and cathodic protection requirements that must be evaluated for each specific application.

Hydraulic Elevator

Implement *all* the following measures:

1. Electrically insulate each cylinder from building metals by installing dielectric material between the piston platen and car, insulating the bolts, and installing an insulated joint in the oil line.
2. Choose one of the following corrosion control options for the hydraulic steel cylinders.

   OPTION 1

   a. Coat hydraulic elevator cylinders as described above for steel pipe, item #4, option 1.

   b. Apply cathodic protection to hydraulic cylinders as per NACE Standard SP0169.

   OPTION 2

   a. As an alternative to electrical insulation and cathodic protection, place each cylinder in a plastic casing with a plastic watertight seal at the bottom.
3. The elevator oil line should be placed above ground if possible but, if underground, should be protected by one of the following corrosion control options:

   OPTION 1

   a. Provide a bonded dielectric coating.

   b. Electrically isolate the pipeline.

   c. Apply cathodic protection to steel piping as per NACE Standard SP0169.

   OPTION 2

   a. Place the oil line in a PVC casing pipe with solvent-welded joints to prevent contact with soil and soil moisture.

Iron Pipe

Implement *all* the following measures:

1. Electrically insulate underground iron pipe from dissimilar metals and from above ground iron pipe with insulating joints per NACE Standard SP0286.
2. Bond all nonconductive type joints for electrical continuity. Electrical continuity is necessary for corrosion monitoring and possible future cathodic protection.
3. Install corrosion monitoring test stations to facilitate corrosion monitoring and the application of possible future cathodic protection:
    a. At each end of the pipeline.
    b. At each end of any casings.
    c. Other locations as necessary so the interval between test stations does not exceed 1,200 feet.
4. Choose one of the following corrosion control options:

    OPTION 1

    a. Apply a suitable coating intended for underground use such as:
        i. Polyethylene encasement per AWWA C105; *or*
        ii. Epoxy coating; *or*
        iii. Polyurethane; *or*
        iv. Wax tape.

        NOTE: The thin factory-applied asphaltic coating applied to ductile iron pipe for transportation and aesthetic purposes does not constitute a corrosion control coating.

    b. Although it is customary to cathodically protect coated structures, cathodic protection is not recommended at this time due to moderately corrosive soils. Joint bonds, test stations, and insulated joints should still be installed and will facilitate the application of cathodic protection in the future if needed to control leaks.

    OPTION 2

    a. As an alternative to coating systems described in Option 1 and possible future cathodic protection, concrete encase all buried portions of metallic piping so that there is a minimum of 3 inches of concrete cover provided over and around surfaces of pipe, fittings, and valves using any type of cement.

Copper Tubing

Implement *all* the following measures:

1. Place cold water copper tubing in an 8-mil polyethylene sleeve or encase in double 4-mil thick polyethylene sleeves and bed and backfill with clean sand at least 2 inches thick surrounding the tubing. Clean sand should have a minimum resistivity of no less than

3000 ohm-cm, and a pH of 6.0–8.0. Copper tubing for cold water can also be treated the same as for hot water.

2. Hot water tubing may be subject to a higher corrosion rate. Protect hot copper tubing by one of the following measures:
    a. Preventing soil contact. Soil contact may be prevented by placing the tubing above ground or encasing the tubing with PVC pipe with solvent-welded joints. *or*
    b. Applying cathodic protection per NACE Standard SP0169. The amount of cathodic protection current needed can be minimized by coating the tubing.

Plastic and Vitrified Clay Pipe

1. No special precautions are required for plastic and vitrified clay piping placed underground from a corrosion viewpoint.

2. Protect all metallic fittings and valves with wax tape per AWWA C217 or epoxy.

All Pipe

1. On all pipes, appurtenances, and fittings not protected by cathodic protection, coat bare metal such as valves, bolts, flange joints, joint harnesses, and flexible couplings with wax tape per AWWA C217 after assembly.

2. Where metallic pipelines penetrate concrete structures such as building floors, vault walls, and thrust blocks use plastic sleeves, rubber seals, or other dielectric material to prevent pipe contact with the concrete and reinforcing steel.

Concrete

1. From a corrosion standpoint, any type of cement may be used for concrete structures and pipe because the sulfate concentration is negligible, 0 to 0.1 percent.[3,4,5]

2. Standard concrete cover over reinforcing steel may be used for concrete structures and pipe in contact with these soils due to the low chloride concentration[6] found onsite.

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[3] *2009 International Building Code (IBC) which refers to American Concrete Institute (ACI-318) Table 4.3.1*
[4] *2009 International Residential Code (IRC) which refers to American Concrete Institute (ACI-318) Table 4.3.1*
[5] *2010 California Building Code (CBC) which refers to American Concrete Institute (ACI-318) Table 4.3.1*
[6] *Design Manual 303: Concrete Cylinder Pipe. Ameron. p.65*

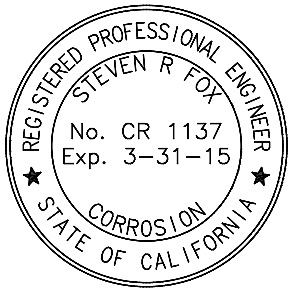
CLOSURE

The analysis and recommendations presented in this report are based upon data obtained from the laboratory sample. This report does not reflect variations that may occur across the site or due to the modifying effects of construction. If variations appear, HDR|Schiff should be notified immediately so that further evaluation and supplemental recommendations can be provided.

Our services have been performed with the usual thoroughness and competence of the engineering profession. No other warranty or representation, either expressed or implied, is included or intended.

Please call if you have any questions.

Respectfully Submitted,
HDR Engineering, Inc.



Anaissa Amador

Steven R. Fox, P.E.

Enc: Table 1

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Table 1 - Laboratory Tests on Soil Samples

Geotechnologies, Inc.
MGA North LLC
Your #20572, HDR|Schiff #13-0572SCS
11-Jul-13

Sample ID			B17 @ 1-5'
Resistivity		**Units**	
as-received		ohm-cm	13,200
saturated		ohm-cm	3,000
pH			7.5
Electrical Conductivity		mS/cm	0.17
Chemical Analyses			
Cations			
calcium	Ca^{2+}	mg/kg	95
magnesium	Mg^{2+}	mg/kg	5.6
sodium	Na^{1+}	mg/kg	133
potassium	K^{1+}	mg/kg	21
Anions			
carbonate	CO_3^{2-}	mg/kg	ND
bicarbonate	HCO_3^{1-}	mg/kg	534
fluoride	F^{1-}	mg/kg	6.4
chloride	Cl^{1-}	mg/kg	4.1
sulfate	SO_4^{2-}	mg/kg	32
phosphate	PO_4^{3-}	mg/kg	ND
Other Tests			
ammonium	NH_4^{1+}	mg/kg	ND
nitrate	NO_3^{1-}	mg/kg	ND
sulfide	S^{2-}	qual	nd
Redox		mV	na

Electrical conductivity in millisiemens/cm and chemical analysis were made on a 1:5 soil-to-water extract.
mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na = not analyzed