E. Geology, Soils and Phase I

Soil Engineering Investigation, C.Y. Geotech., Inc.

Phase 1 Environmental Site Assessment, Dominion Due Diligence Group

C. Y. GEOTECH, INC.
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SOIL ENGINEERING INVESTIGATION

PROPOSED ONE-STORY TO SIX-STORY MIXED USE BUILDINGS
WITH THREE LEVELS OF SUBTERRANEOUS PARKING
13005 - 13609 VICTORY BOULEVARD, VAN NUYS, CALIFORNIA

FOR

MR. GILBERT SALAZAR

AUGUST 31, 2007
PROJECT NO. CYG-07-4948

August 31, 2007 P.N. CYG-07-4948

Mr. Gilbert Salazar
6014 Greenbush Avenue
Sherman Oaks, CA 91401

Subject: Soil Engineering Investigation, Proposed One-Story to Six-Story Mixed Use Building With Three Levels of Subterraneous Parking, 13005 - 13069 Victory Boulevard, Van Nuys, California

Dear Mr. Salazar,

Per your request, C. Y. Geotech (CYG), Inc. has performed a soil engineering investigation for the subject project. The purposes of this investigation are to evaluate the engineering properties of onsite earth materials which may affect the proposed development and to provide recommendations for the design and construction of the proposed mixed use building. The accompanying report presents the findings and conclusions of this investigation and the recommendations for the design and construction of the proposed mixed use buildings.

Based upon the findings of this investigation, the development of the proposed mixed use buildings at the subject site is feasible from a geotechnical engineering viewpoint provided the recommendations of the accompanying report are incorporated into design and implemented during construction. Conventional spread footings, mat foundation or deep foundation such as skin friction piles entirely founded into competent alluvium can be used to support the proposed mixed use buildings.

Shoring system will be required for deep temporary excavations. Soldier piles with tieback-lagging system can be used for shoring of temporary excavations. A monitoring program for lateral displacement of soldier piles will be required for the shoring system.

Provided the recommendations in this report are properly incorporated into design and implemented during construction, the proposed mixed use buildings will be safe from geologic hazards including settlement, landsliding, slippage and liquefaction and the development of the proposed mixed condominium/retailer buildings will not adversely affect the geologic stability of the site and adjacent properties.

We appreciate the opportunity for providing the professional service. If you have any questions regarding this report, please do not hesitate to contact us.

Very truly yours,
C. Y. Geotech, Inc.

John T. Tsao
RCE 46886/CEG 1783

Encl: Appendix A, Field Exploration and Laboratory Testing
Appendix B, Liquefaction Evaluation

cc: (5) Addressee

SOIL ENGINEERING INVESTIGATION
Proposed One-Story to Six-Story Mixed Use Buildings
With Three Levels of Subterraneous Parking
13005 - 13609 Victory Boulevard, Van Nuys, California

As requested, CYG has performed a soil engineering investigation for the subject project. The purposes of this investigation are to evaluate the engineering properties of onsite earth materials which may affect the proposed development and to provide recommendations for the design and construction of the proposed one-story to six-story mixed use buildings.

1.0 SCOPE OF WORK

The following field, laboratory and engineering works have been performed for this investigation:

a. Data research and review of available geologic and geotechnical engineering data of the site and its vicinity. A site location map is shown on Figure 1.

b. Logging and sampling eight (8) deep borings to a maximum depth of 85 feet at the location as shown on Plate 1 for liquefaction and foundation evaluation.

c. Perform laboratory tests to determine the engineering properties of onsite earth materials. The results of laboratory tests are presented in Appendix A and summarized in Section 6.3.

d. Perform faulting study and seismic evaluation. The potential of earthquake-induced geologic hazards which may affect the stability of the site was evaluated. The liquefaction potential of onsite soils was evaluated. The building code seismic factors for structural design were determined.

e. Perform soils engineering evaluation and analyses. Slope stability analyses were performed to calculate the equivalent fluid pressures and lateral force required for the design of basement wall and to evaluate the stability of temporary excavations.

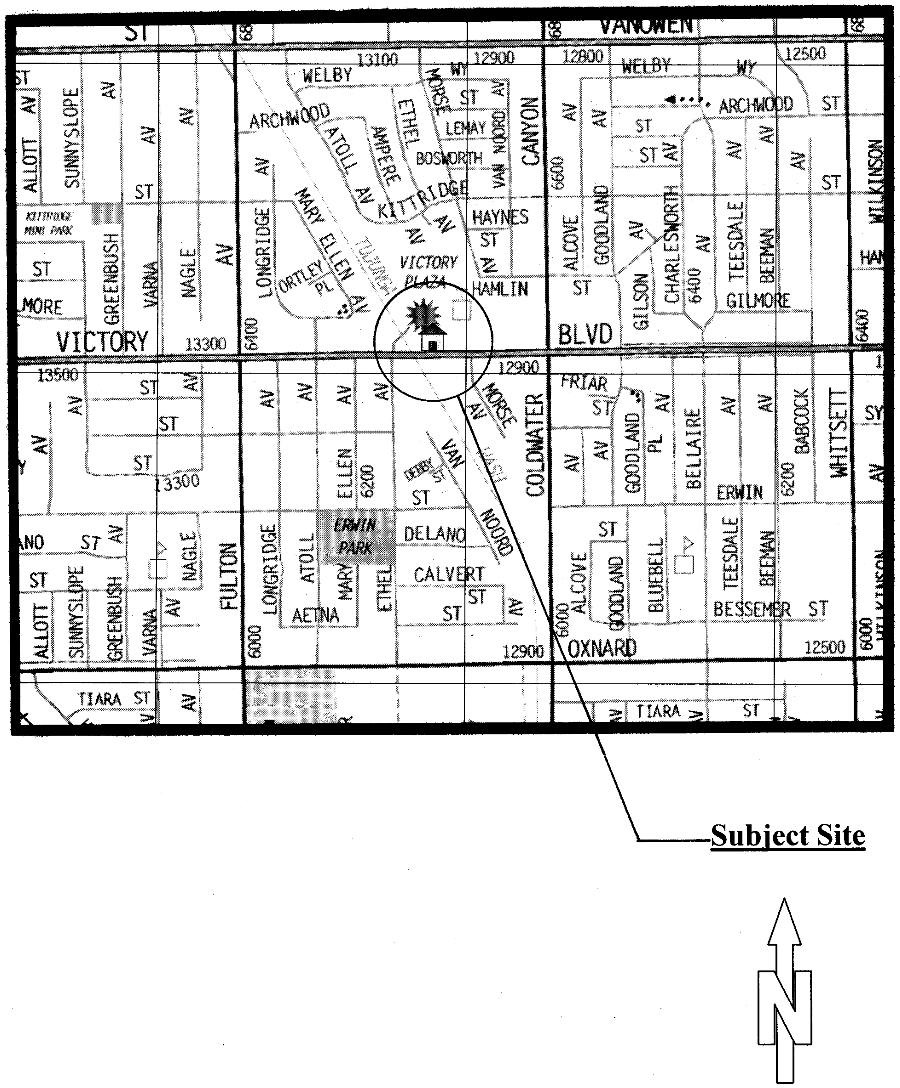
f. Prepare this report to present the findings and conclusions of this investigation and to provide recommendations for the design and construction of the proposed mixed use building.

2.0 SITE DESCRIPTION

The subject site is located at 13005 - 13609 Victory Boulevard, Van Nuys, California. A site location map is shown on Figure 1. The site is bounded on the south by Victory Boulevard, on the southwest by Tujunga Wash and on other sides by commercial and residential buildings. The site is roughly trapezoidal-shaped, fairly level and currently occupied by commercial buildings and parking lots. A site plan showing the site, the property lines, the existing structures and the proposed mixed use buildings is shown on Plate 1. Three geotechnical cross sections (A-A', B-B' and C-C') showing the site, the immediate adjacent properties and the earth materials underlying the site are shown on Plate 2.

3.0 PREVIOUS INVESTIGATION AND PAST GRADING

Data research of grading records, soil reports and geologic reports for the subject site was performed by one of our engineers at the City of Los Angeles Department of Building and Safety. However, no soils report or geologic report for the past development of the site was found from the city files during our data research.



Scale 1"= 1200'

(Adopted from Thomas Brothers CD-ROM, 1999)

C. Y. GEOTECH, INC. Engineering Geology and Geotechnical Engineering	Site Location Map	
	CYG-07-4948	Figure 1

4.0 PROPOSED DEVELOPMENT

Information regarding the proposed development was provided by you and was used as a guide for the field exploration and report preparation. It is our understanding that the existing building structures are to be demolished and one-story, three-story, five-story and six-story mixed use buildings are to be built on th site. The buildings will be provided by a three levels of parking basement under the entire site. A site plan showing the site, the property lines and the proposed mixed use buildings is shown on Plate 1. Three geotechnical cross sections (A-A', B-B' and C-C') showing the proposed buildings and the subsurface conditions of the site and immediate adjacent sites are shown on Plate 2.

Formal grading, architectural and structural plans have not been prepared and await the findings, conclusions and recommendations of this investigation.

5.0 FIELD EXPLORATION AND LABORATORY TESTING

Field exploration was performed by one of our engineers on June 11 and 12, 2007 with the aid of a hollow-stem drill rig. Eight (8) deep borings were drilled to a maximum depth of 85 feet at the locations as shown on Plate 1 for liquefaction and foundation evaluations. The borings were logged by the engineer and backfilled on the same day of drilling. The boring logs are presented in Appendix A.

The earth materials encountered in the borings were sampled by using a split-tube soil sampler and a SPT soil sampler. The SPT soil samples were collected by using a 140-pound hammer to drive the SPT standard tube 18 inches into the soil. The falling head for SPT hammer was 30 inches. The blow count values were taken for every 6-inch penetration. The total blow count for the last 12 inches of penetrating distance was recorded as SPT N value. The SPT samples of onsite earth materials were logged and then retained in plastic bags for laboratory particle size tests.

The ring samples of onsite earth materials were logged and then retained in a series of brass rings, each having an inner diameter of 2.4 inches and a height of 1 inch. The ring samples and brass rings were retained in plastic, close-fitting, moisture-tight containers. A bulk sample of onsite soil was collected for laboratory compaction test and expansion index test.

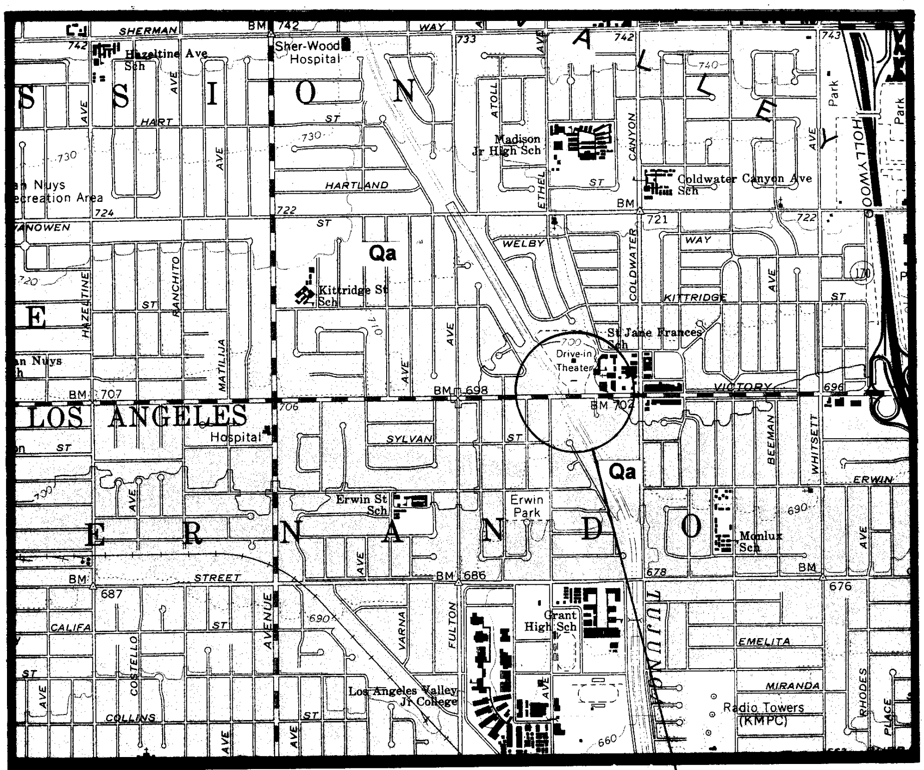
Laboratory testing was performed after the review of field data and in consideration of the proposed development and the probable foundation and footing system to be utilized. The testing procedures of ASTM (American Society for Testing and Materials) Standards were followed in laboratory testing. The following engineering properties of onsite earth materials were determined: 1) field density and field moisture content, 2) maximum dry density and optimum moisture content, 3) cohesion and friction angle, 4) compressibility and hydroconsolidation, 5) expansion index test, and 6) particle size test. The results of laboratory tests are presented in Appendix A and summarized in Section 6.3.

6.0 EARTH MATERIAL

The earth materials encountered in the borings consisted of artificial fill and alluvium. The alluvial soil observed within the site is consistent with the Dibblee Geologic Map (see Figure 2). Descriptions of the soils encountered in the borings are shown on the boring logs enclosed in Appendix A. The engineering properties of onsite soils determined from laboratory tests are presented in Appendix A and summarized in Section 6.3.

6.1 Artificial Fill (Af)

Artificial fill was encountered from the ground surface and to a depth of 1 foot in boring B-6. The fill soil consists of light brown gravelly silty sand in a slightly moist and moderately dense condition. The fill soil, at its present condition, is not suitable to be used for foundation and slab support.



Subject Site

Legend

Qa	SURFICIAL SEDIMENTS *alluvial gravel, sand, and silt-clay, derived mostly from Santa Monica Mountains; includes gravel and sand of stream channels*

N

Scale 1" = 2000'

(Adopted from Geologic Map of the Beverly Hills-Van Nuys (South 1/2) Quadrangle, Los Angeles County, California, Dibblee, 1991)

C. Y. GEOTECH, INC. Engineering Geology and Geotechnical Engineering	**Regional Geologic Map**	
	CYG-07-4948	**Figure 2**

6.2 Alluvium (Qa)

Alluvium was encountered underlying fill soil and to the depth explored in boring B-2 and from the ground surface to the depth explored in all other borings. The alluvial soil consists primarily of light brown and grayish brown silty sand, sand gravelly sand and brown clayey silty sand and silty sand. The alluvial soil observed was in a slightly mist to moist and moderately dens to dense condition.

The laboratory moisture-density tests indicated that the dry density of the alluvial soil is in the range of 91 to 136 pounds per cubic foot (pcf). The laboratory expansion index test indicated expansion indexes of 3, 4 and 26 for the tested alluvial soil. A soil with an expansion index in the range of 0 to 20 is classified as a very low expansive soil. A soil with an expansion index in the range of 21 to 50 is classified as a low expansive soil.

6.3 Engineering Property

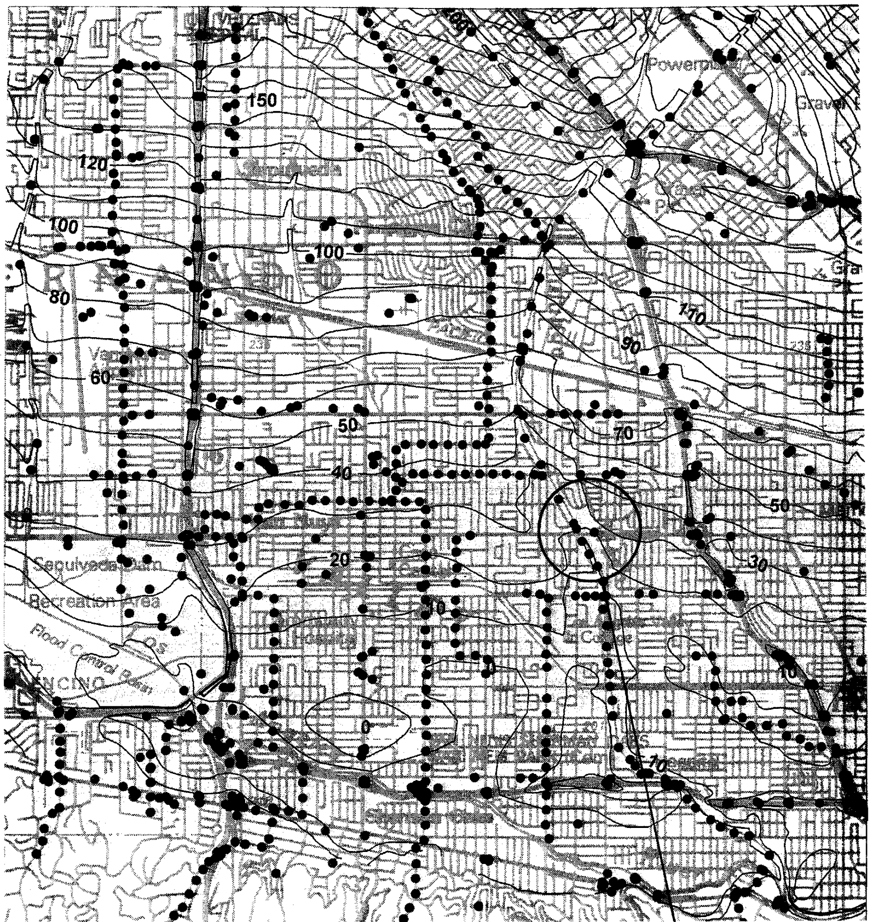
The engineering properties of onsite soils determined from laboratory tests are summarized below:

Field Dry Density (Qa):	91 - 136 pcf
Field Moisture Content (Qa):	1 - 22 %
Maximum Dry Density (Qa, silty sand and gravelly sand):	126.5 pcf
Maximum Dry Density (Qa, clayey sandy silt):	132 pcf
Optimum Moisture Content (Qa, silty sand and gravelly sand):	7 & 9 %
Optimum Moisture Content (Qa, clayey sandy silt):	9 %
Cohesion (Qa, peak):	60 - 320 psf
Cohesion (Qa, ultimate):	30 - 260 psf
Friction Angle (Qa, peak):	25 - 39.5 deg
Friction Angle (Qa, ultimate):	25.5 - 39 deg
Compressibility (Qa):	See Plates CS-1 to CS-27
Hydroconsolidation (Qa):	0 - 1.1 % (Average = 0.28%)
Expansion Index (Qa, sand and gravelly sand):	E.I. = 3 & 4
Expansion Index (Qa, clayey sandy silt):	E.I. = 26

6.4 Surface Water and Subsurface Water

No surface water was observed within the site during our field exploration. Surface water within the site is limited to the precipitation falling directly on the site.

No groundwater was encountered in the borings to the depth explored. As shown on the CDMG (California Division of Mines and Geology) Historically Highest Groundwater Map (see Figure 3), the historical highest ground water underlying the site is approximately 20 feet. In our opinion, the groundwater underlies the site at depth and does not appear to be close enough to the surface to significantly affect the stability of the site and proposed development.



C. Y. GEOTECH, INC. Engineering Geology and Geotechnical Engineering	Historically Highest Ground Water Contours and Borehole Log Data Locations	
	CYG-03-4948	Figure 3

7.0 FAULTING AND SEISMICITY STUDY

The computer programs of EQFAULT and FRISK89 were used in the faulting and seismicity studies. EQFAULT is a computer program for the deterministic prediction of peak horizontal acceleration from digitized California faults. FRISK89 is a computer program for the probabilistic estimation of peak acceleration and uniform hazard spectra using 3-D faults as earthquake sources.

7.1 Faulting Study

The faulting study indicated that the site is not located within any of the mapped Alquist-Priolo Special Studies Zones and no fault trace of any known active or potentially active fault passes through the site. However, the site, as all of the Southern California areas, is located within a seismically active region and will experience slight to very intense ground shaking as the result of movement along various active faults in the region. Thirty (30) fault systems are located within a search radius of 50 miles from the site. The fault systems which are near the site and may significantly affect the stability of the site are Verdugo fault, Northridge Hills fault, Sierra Madre-San Fernando fault, Santa Monica-Hollywood fault, Newport-Inglewood fault, Elysian Park Seismic fault zone, Santa Susana fault, San Gabriel fault and Raymond fault.

The Alquist-Priolo Special Studies Zones Act was signed into law on December 22, 1972, and went into effect in March of 1973. The purpose of this Act is to prohibit the location of most structures for human occupancy across the traces of active faults and to mitigate thereby the hazard of fault-rupture. The development permits for development projects within the special study zones will be withheld by the city or county until geologic investigations demonstrate that the sites are not threatened by surface displacement from future faulting.

7.2 Seismicity Study

The seismicity study indicated that the largest credible and probable peak ground accelerations (mean (m) + 1 standard deviation (σ)) which may impact the site are 0.76g (g:gravity) and 0.40g, respectively. The largest credible and probable repeatable high ground accelerations (m+σ) which may impact the site are 0.49g and 0.26g, respectively. The peak and repeatable high ground accelerations (m+σ) for a 50-year exposure and 10% exceedance are approximately 0.52g and 0.34g, respectively. The maximum credible magnitude, credible peak ground acceleration and credible repeatable high ground acceleration which may impact the subject site caused by the most significant fault systems are shown in the following table.

Fault Name	Distance from the Site, km	Maximum Credible Magnitude	Maximum Credible Peak Ground Acceleration	Maximum Credible Repeatable High Ground Acceleration
Verdugo fault	5	6.7	0.76g	0.49g
Northridge Hills fault	7	6.5	0.64g	0.42g
Sierra Madre-San Fernando fault	11	7.5	0.71g	0.46g
Santa Monica-Hollywood fault	13	7.5	0.66g	0.43g
Newport-Inglewood fault	15	7.0	0.38g	0.25g
Elysian Park Seismic fault zone	15	7.0	0.46g	0.30g
Santa Susana fault	15	7.0	0.46g	0.30g
San Gabriel fault	18	7.0	0.33g	0.21g
Raymond fault	19	7.5	0.51g	0.33g

7.3 Uniform Building Code Seismic Factors

As shown on Figure 4, the site is located close to 5-kilometer near-source zones of the Verdugo fault. The seismic factors listed in the following table for structural design of the proposed apartment building were determined based on the findings of data research and seismic evaluation and in accordance with California Building Code, Uniform Building Code and CDMG Active Fault Near-Source Zone Maps.

Seismic Factor	Value	Reference
Seismic Zone	Zone 4	Figure 16-2, 1997 UBC
Seismic Zone Factor	0.40	Table 16-I, 1997 UBC
Soil Profile Type	Sd	Table 16-J, 1997 UBC
Seismic Source Type (Verdugo fault)	B	Table 16-U, 1997 UBC
Near Source Factor, Na (Verdugo fault)	1.0	Table 16-S, 1997 UBC
Near Source Factor, Nv (Verdugo fault)	1.2	Table 16-T, 1997 UBC
Seismic Response Coefficient, Ca	0.44 Na	Table 16-Q, 1997 UBC
Seismic Response Coefficient, Cv	0.64 Nv	Table 16-R, 1997 UBC

7.4 UBC Design Response Spectrum

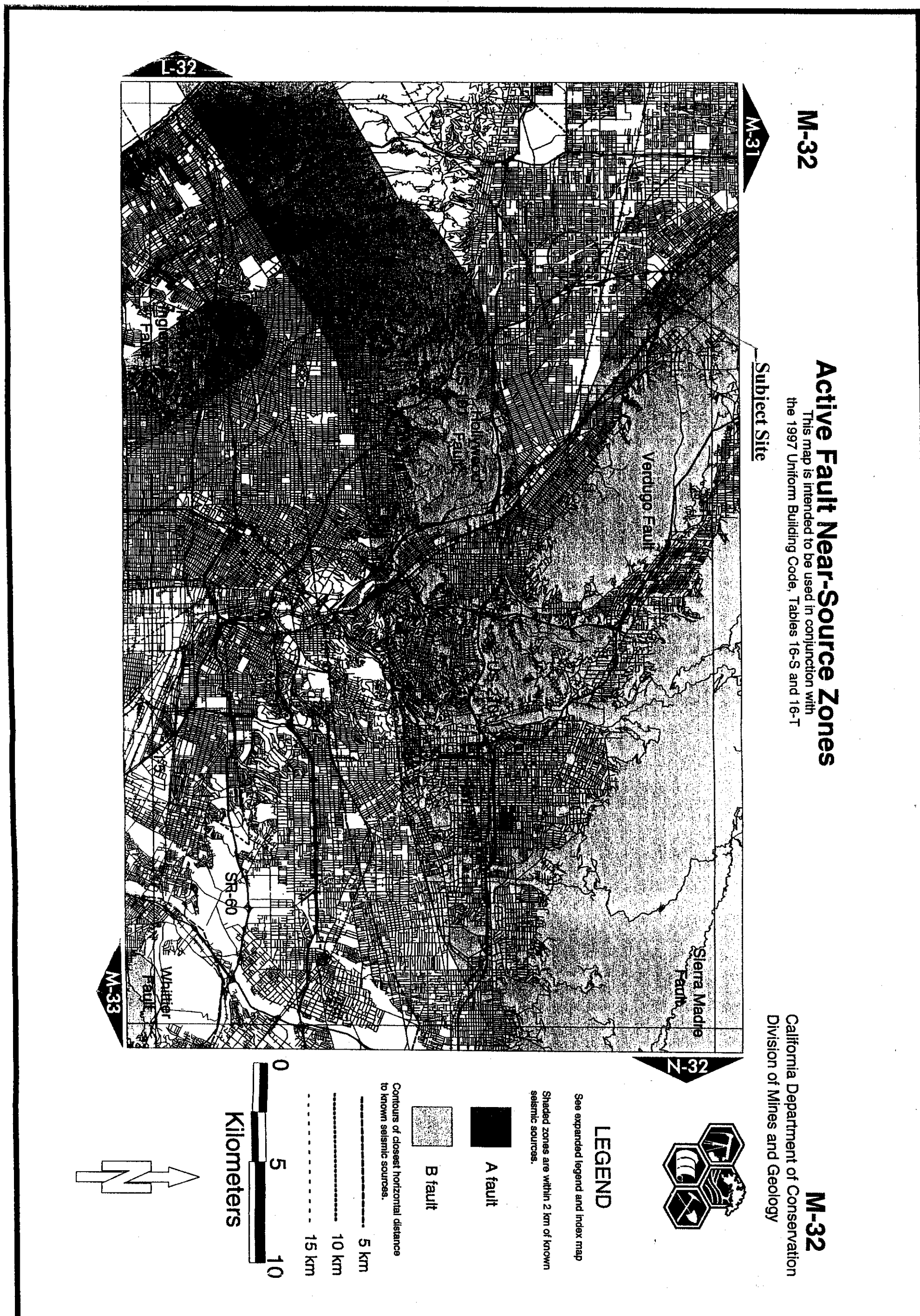
The dynamic approach of the Uniform Building Code (UBC) allows the response of the structure to be determined by response spectrum analysis. For design purposes, the response spectrum should be representative of the characteristics of all seismic properties experienced at a specific site. The design response spectrum should be based in geologic, tectonic, seismological, and soil characteristics associate with that specific site if these are known. If not, it may be constructed according to the spectral shape presented by UBC-97 Figure 16-3.

An elastic design response spectrum constructed in accordance with the site-specific values of seismic response coefficients, Ca and Cv is shown on Figure 5. The design acceleration ordinates shall be multiplied by the acceleration of gravity, 386.4 in/sec^2 (UBC-97, Section 1631.2.1).

The vertical component of ground motion may be defined by scaling corresponding horizontal accelerations by a factor of two-thirds. Alternative factors may be used when substantiated by site specific data. Where the Near Source Factor, Na, is greater than 1.0, site-specific vertical response spectra shall be used in lieu of the factor of two-thirds (UBC-97, Section 1631.2.5).

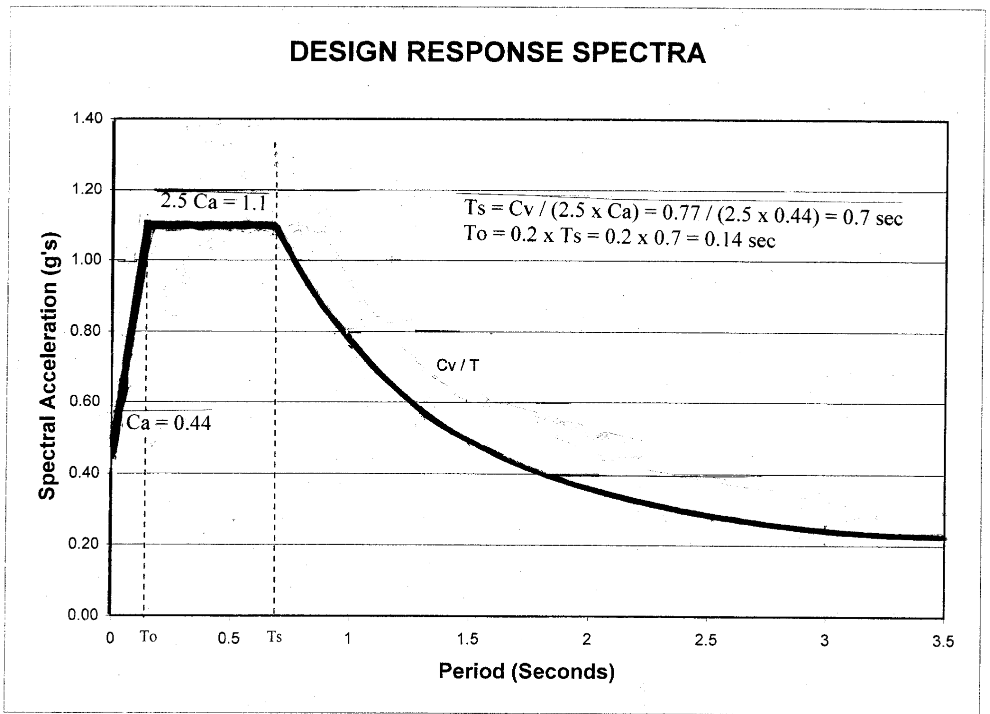
7.5 Newmark-Hall Design Response Spectrum

The ground motion values presented in Section 1.0 were used as a basis to compute the site dependent response spectra. The technique introduced by N. M. Newmark and W. J. Hall (1969) was used to generate of the tripartite plot of the design response spectrum. The calculations of the ground acceleration, ground velocity and ground displacement for 5%, 10%, 15% and 20% damping ratio are shown on Figure 6 and Figure 7. The peak ground acceleration ($m+\sigma$) of 0.61g for a 100-year exposure and 10% exceedance and the peak ground acceleration ($m+\sigma$) of 0.52g for a 50-year exposure and 10% exceedance were used in the calculations of ground acceleration, ground velocity and ground displacement. The response spectra for different damping ratio can be determined by using different amplification factors as shown on Figure 6 and Figure 7.



Active Fault Near-Source Zone		C. Y. GEOTECH, INC. Engineering Geology and Geotechnical Engineering
CYG-07-4948	Figure 4	

Soil Type	Sd
Na	1.0
Nv	1.2
Ca	0.44
Cv	0.77



C. Y. GEOTECH, INC. Engineering Geology and Geotechnical Engineering	Design Response Spectra	
	CYG-03-4948	Figure 5

BROAD-BANDED DESIGN SPECTRA WITH NEWMARK-HALL METHOD (TRIPARTITE PLOT)

Project: Salazart / Victory / 50yr exposure & 10% exceedance

Purpose: Develop a Newmark-HallDesign Spectrum for 5%, 10%, 15% and 20% damping

Table 1. Amplification Factors for Newmark-Hall Design Spectra

Structural Damping Ratio	0	2	5	10	15	20
Displacement	2.5	18	1.4	1.1	1.0	1.0
Velocity	4.0	2.8	1.9	1.3	1.2	1.1
Acceleration	6.4	4.3	2.6	1.5	1.3	1.2

Fault name: Sierra Madre-San Fernando fault
Distance From the Site: 11 km
Maximum Credible Magnitude: 7.5
Peak High Ground Accelerations (m+s) for a 50 year exposure and 10% exceedance = 0.52g

<u>Maximum Ground Motion Parameters</u>
Maximum Ground Acceleration: Sa = (0.52) x (1.0 g) = 0.52g
Maximum Ground Velocity: Sv = (0.52) x (48 in/sec) = 25.0 in/sec
Maximum Ground Displacement: Sd = (0.52) x (36 in) = 18.7 in

<u>Amplified Response Parameters for 5% Damping</u>
Amplified Acceleration: Sa = (2.6) x (0.52 g) = 1.59g
Amplified Velocity: Sv = (1.9) x (25.0 in/sec) = 47.5 in/sec
Amplified Displacement: Sd = (1.4) x (18.7 in) = 26.2 g

<u>Amplified Response Parameters for 10% Damping</u>
Amplified Acceleration: Sa = (1.5) x (0.52 g) = 0.78g
Amplified Velocity: Sv = (1.3) x (25.0 in/sec) = 32.5 in/sec
Amplified Displacement: Sd = (1.1) x (18.7 in) = 20.6 g

<u>Amplified Response Parameters for 15% Damping</u>
Amplified Acceleration: Sa = (1.3) x (0.52 g) = 0.68g
Amplified Velocity: Sv = (1.2) x (25.0 in/sec) = 30.0 in/sec
Amplified Displacement: Sd = (1.0) x (18.7 in) = 18.7 g

<u>Amplified Response Parameters for 20% Damping</u>
Amplified Acceleration: Sa = (1.2) x (0.52 g) = 0.62g
Amplified Velocity: Sv = (1.1) x (25.0 in/sec) = 27.5 in/sec
Amplified Displacement: Sd = (1.0) x (18.7 in) = 18.7 g

Figure 6

BROAD-BANDED DESIGN SPECTRA WITH NEWMARK-HALL METHOD (TRIPARTITE PLOT)

Project: Salazart / Victory / 100yr exposure & 10% exceedance

Purpose: Develop a Newmark-HallDesign Spectrum for 5%, 10%, 15% and 20% damping

Table 1. Amplification Factors for Newmark-Hall Design Spectra

Structural Damping Ratio	0	2	5	10	15	20
Displacement	2.5	18	1.4	1.1	1.0	1.0
Velocity	4.0	2.8	1.9	1.3	1.2	1.1
Acceleration	6.4	4.3	2.6	1.5	1.3	1.2

Fault name: Sierra Madre-San Fernando fault
Distance From the Site: 11 km
Maximum Credible Magnitude: 7.5
Peak High Ground Accelerations (m+s) for a 50 year exposure and 10% exceedance = 0.61g

Maximum Ground Motion Parameters
Maximum Ground Acceleration: Sa = (0.61) x (1.0 g) = 0.61g
Maximum Ground Velocity: Sv = (0.61) x (48 in/sec) = 29.3 in/sec
Maximum Ground Displacement: Sd = (0.61) x (36 in) = 22.0 in

Amplified Response Parameters for 5% Damping
Amplified Acceleration: Sa = (2.6) x (0.61 g) = 1.59g
Amplified Velocity: Sv = (1.9) x (29.3 in/sec) = 55.7 in/sec
Amplified Displacement: Sd = (1.4) x (22.0 in) = 30.8 g

Amplified Response Parameters for 10% Damping
Amplified Acceleration: Sa = (1.5) x (0.61 g) = 0.91g
Amplified Velocity: Sv = (1.3) x (29.3 in/sec) = 38.1 in/sec
Amplified Displacement: Sd = (1.1) x (22.0 in) = 24.2 g

Amplified Response Parameters for 15% Damping
Amplified Acceleration: Sa = (1.3) x (0.61 g) = 0.79g
Amplified Velocity: Sv = (1.2) x (29.3 in/sec) = 35.2 in/sec
Amplified Displacement: Sd = (1.0) x (22.0 in) = 22.0 g

Amplified Response Parameters for 20% Damping
Amplified Acceleration: Sa = (1.2) x (0.61 g) = 0.73g
Amplified Velocity: Sv = (1.1) x (29.3 in/sec) = 32.2 in/sec
Amplified Displacement: Sd = (1.0) x (22.0 in) = 22.0 g

Figure 7

7.6 Structural Period

The fundamental period of the structure should be determined by rational methods as introduced as Method B in UBC Section 1630.2.2. However, due to the formal structural plans have not been prepared, the exact height, design loads of the mixed use buildings, and the construction material, stiffness and strength of structure are unknown at this time of investigation. Therefore, the structure period is prepared based on Method A, an approximated approach in UBC Section 1630.2.2. It is approximated by

$$T = C_t(h_n)^{3/4}$$

Where C_t has values of 0.035 for steel moment-resisting frames, 0.030 for reinforced concrete moment-resisting frames and eccentrically braced steel frames, and 0.020 for all other structures, and h_n is the height (in feet) of the uppermost level of the main portion of the structure above the base. A 70-foot height, steel moment-resisting frames is assumed in the calculation. The structure period was determined to be 0.85 second for a 70-foot high building.

Structural Engineering should determine the Structure Period using rational method such as by using Method B. The value of the structure period obtained from Method B shall not exceed a value 30 percent greater than the value of structure period obtained from Method A in Seismic Zone 4.

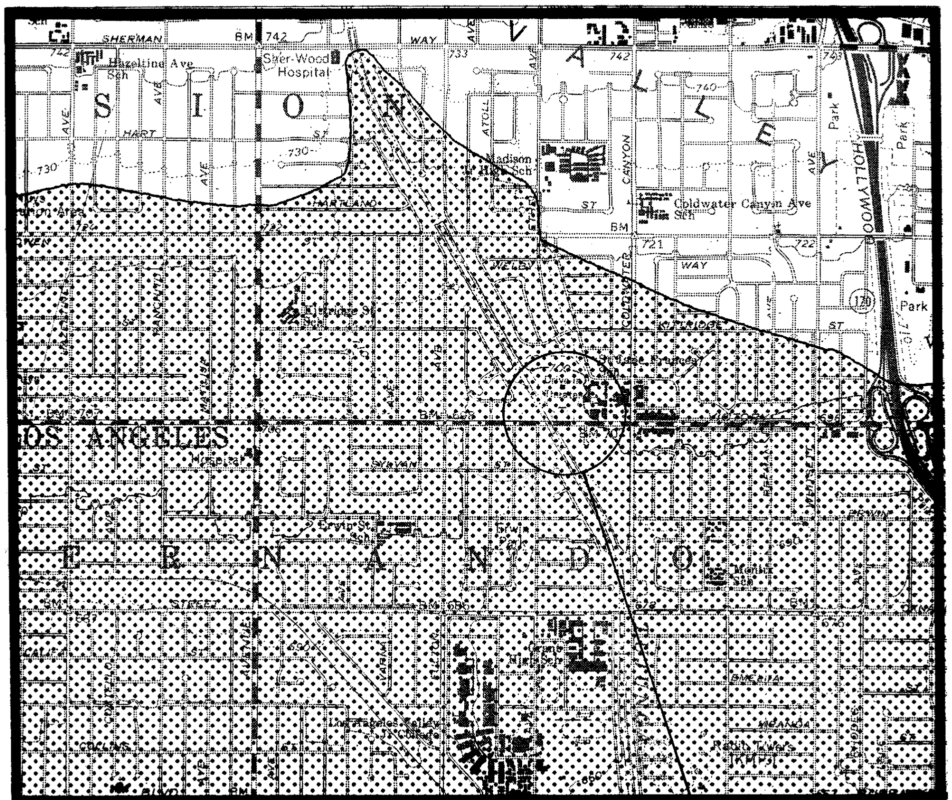
7.7 Liquefaction

Liquefaction describes a phenomenon in which cyclic stresses produced by ground shaking induced excess pore water pressures in the cohesionless soils. These soils may thereby acquire a high degree of mobility leading to damaging deformations. In general, this phenomenon only occurs below the water table, but after liquefaction has developed, it can propagate upward into overlying non-saturated soil as excess pore water pressure. In general, liquefaction has three major effects: 1) the consolidation of loose sediments with resultant settlement of the ground surface, 2) lateral sliding or spreading, and 3) sand boiling.

Liquefaction susceptibility under a given earthquake is related to the gradation and relative density characteristics of the soil, the in-situ stresses prior to ground motion, and the depth to the water table, as well as other factors. A site that is susceptible to liquefaction should have the following four principal conditions: 1) the site is located within a seismically active zone, 2) the site should have layers of soils that are cohesionless and contain less than 15% of clay size particles, 3) groundwater exists within 50 feet of the ground surface or records indicate that the recent water table has been higher than 30 feet or there is a likelihood that groundwater will rise above 50 feet, and 4) soil should have relative densities between 50% to 70%.

As shown on Figure 8, the site is located within one of the liquefaction susceptible zones as mapped in the CDMG Seismic Hazards Maps. Therefore, a liquefaction evaluation was performed for the subject site. The maximum credible magnitude of the Sierra Madre-San Fernando fault and the peak ground acceleration ($m+\sigma$) for a 50-year exposure and 10% exceedance were used in the liquefaction evaluation. The seismic parameters used in liquefaction evaluation are shown in the following table.

Fault Simulated	Maximum Credible Magnitude	Peak Horizontal Ground Acceleration 50-year exposure and 10% exceedance
Sierra Madre-San Fernando	7.5	0.52g



Subject Site

Legend

Liquefaction
Areas where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such as that mitigation as defined in Public Resources Code section 2693 (c) would be required

Earthquake-Induced Landslide
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code section 2693 (c) would be required

Scale 1" = 2000'

(Adopted from State of California Seismic Hazard Zones.)

C. Y. GEOTECH, INC. Engineering Geology and Geotechnical Engineering	**Seismic Hazard Map**
CYG-07-4948	**Figure 8**

The liquefaction evaluation method introduced by Seed and Idriss (1982) was used in the calculation of the factor of safety for liquefaction potential. The factor of safety is defined as the ratio of the cyclic stress ratio to cause liquefaction to the earthquake-induced cyclic stress ratio. When the factor of safety exceeds the high end of the empirical range, the factor of safety is defined as "Infinite".

Although no groundwater was encountered in the borings to the depth explored (80 feet), the CDMG Historically Highest Groundwater Map indicates a historical highest ground water of approximately 20 feet underlying the site. Therefore, a groundwater table of 20 feet below the existing ground surface was used in the liquefaction evaluation.

Due to the proposed three subterraneous parking levels, the following two cases were assumed in the liquefaction evaluation: 1) overburden pressure was calculated starting from the existing ground surface and 2) overburden pressure was calculated from the bottom of subterraneous parking levels.

The results of the liquefaction evaluation are presented in Appendix B and summarized in the following table. The evaluation indicates that the occurrence of liquefaction within onsite soils is unlikely due to either high SPT blow count, high clay content or above groundwater.

Depth (ft)	Water Table =20 ft Overburden from depth = 0 ft	Water Table =20 ft * Overburden from depth = 30 ft	Reasons why no susceptible to liquefaction
0' - 20'	Not Susceptible	---- / above basement	above groundwater
20' - 30'	Not Susceptible	---- / above basement	high SPT blow count and/or clay content
30' - 80'	Not Susceptible	Not Susceptible	high SPT blow count and/or clay content

* Overburden was calculated from the bottom of basement
* Groundwater surface was assumed at the bottom of basement

7.8 Earthquake-Induced Geologic Hazards

Based on the findings of the faulting and seismicity evaluation, it is our opinion that the occurrence of earthquake-induced geologic hazards such as lurching, shallow ground rupture, landslide and liquefaction within the site is unlikely. If a strong earthquake occurs in the vicinity of the subject site, structural distress and minor foundation disturbance caused by earthquake induced shaking will be the major causes of damage. The potential of liquefaction within onsite soil is discussed in section 7.7. Other potential of geologic hazards which may affect the stability of the site are discussed in the following subsections.

7.8.1 Potential of Shallow Ground Rupture

Ground rupture describes a phenomenon in which a gap or rupture of the ground surface occurs during earthquake movement along the intersection of the upper edge of the fault zone and the ground surface.

As addressed in Section 7.1, the subject site is not located within any of the mapped Alquist-Priolo Special Studies Zones and no fault trace of any known active or potentially active fault crosses the site. In our opinion, the potential of ground rupture or cracking within the site due to shaking from local seismic events is low.

7.8.2 Landsliding and Lateral Spreading

Earthquake-induced landsliding describes a phenomenon in which slopes fails or distress during earthquake shaking. Earthquake-induced lateral spreading describes a phenomenon in which ground surface has lateral movement during earthquake shaking. Lateral spreading can act as a subsequent phenomenon of liquefaction.

The site is essentially a flat site and, therefore, is not subjected to earthquake-induced landsliding. The liquefaction evaluation indicated that the site is not susceptible to liquefaction and, therefore, not subjected to earthquake-induced landsliding or lateral spreading.

7.8.3 Ground Lurching

Ground lurching is defined as earthquake motion at right angles to nature or artificial slopes that results in a series of more or less parallel cracks separating the ground into rough blocks. Lurching is also sometimes used to describe undulating surface waves in the soils. Materials which are most susceptible to lurching effects are unconsolidated with low cohesion. Cracking of the ground surface due to shaking from distant seismic events is not considered a significant hazard, although it is a possibility at any site.

The site is essentially a flat site and, therefore, is not subjected to earthquake-induced ground lurching parallel to the slope. Suitable site processing can eliminate compressible materials of low relative density and, thereby, will tend to reduce the potential for ground lurching.

7.8.4 Seiches and Tsunamis

Seiches are an oscillation of the surface of an inland body of water that varies in period from a few minutes to several hours. Seismic excitations can induce such oscillations. Tsunamis are large sea waves produced by submarine earthquakes or volcanic eruptions.

Since the site is not located close to an inland body of water and is at an elevation sufficiently above sea level to be outside the zone of a tsunami runup, the risk of these two hazards is not pertinent to this site.

7.8.5 Settlement Due to Seismic Shaking

Granular soils are considered susceptible to earthquake-induced settlement, whether the soils are saturated or dry. The potential and amount of earthquake-induced settlement will be affected by the magnitude of earthquake, the strength of soils and the occurrence of groundwater.

Although the majority of onsite soil is granular soil, the magnitude of earthquake-induced settlement will not significantly affect the integrity and competency of the building structure due to the following reasons: 1) the potential of soil liquefaction within the site is low, 2) the building structure will be provided with three levels of subterraneous parking levels, and 3) onsite soil below the foundation level is competent with low compressibility.

8.0 SLOPE STABILITY

The site is fairly level and free from the potential of landslide. As shown on Figure 8, the site is not located within any of the earthquake-induced landslide zones mapped in the CDMG Seismic Hazard Maps. No evidence of deep-seated slope failure or other types of slope failure was observed within the site during our field exploration. No landslide was mapped within the site or in site vicinity in the published geologic map.

Three wedge slope stability analyses using the Freebody Diagram method were performed to determine the equivalent fluid pressures for the design of 10-foot, 20-foot and 30-foot high basement retaining walls. The ultimate shear strength parameters of onsite alluvial soil were used in analyses. The results of the analyses are presented in Figures 9, 10 and 11. The analyses indicated that the triangular-distributed equivalent fluid pressures and trapezoidal-distributed lateral forces listed in the following table can be used in the design of the subterraneous building walls. Subterraneous building walls are usually designed as restrained building walls using trapezoidal-distributed lateral forces.

Equivalent Fluid Pressure (Free Body Diagram Method)

Program Made by C. Y. Geotech, Inc.

Project Name:

CYG-07-4948 10 feet Retaining Wall / Level Backfill (Soil)

GEOMETRY OF CRITICAL ACTIVE WEDGE:

Height of the Retaining Wall	=	10 feet
Slope Angle of Retained Slope	=	0 degree
Dip Angle of Critical Wedge	=	56 degree

SHEAR STRENGTH PARAMETERS:

Unit Weight	=	135 pcf
Cohesion	=	90 psf
Friction Angle	=	30.5 degree
Mobilized Cohesion	=	60.0 psf
Mobilized Friction Angle	=	21.5 degree

REQUIRED FACTOR OF SAFETY = 1.5

RESULTS

Dip Angle of Critical Slip Surface	=	56 degree
Total Weight of Active Wedge	=	4553 lbs
Frictional Resistance (Cm * L)	=	724 lbs
Required External Force for Wall	=	2318 lbs
Required Equivalent Fluid Pressure	=	46.4 psf/ft

** Rankine Wedge is not the most critical wedge **

RECOMMENDED EFP VALUE:

Triangular-Distributed EFP = 46.4x(1+sin(30.5))=70psf/ft

Trapezoidal-Distributed LF = EFP(Tri)/1.6
= 44H psf/ft

Figure 9

Equivalent Fluid Pressure (Free Body Diagram Method)

Program Made by C. Y. Geotech, Inc.

Project Name:
CYG-07-4948 20 feet Retaining Wall / Level Backfill (Soil)

GEOMETRY OF CRITICAL ACTIVE WEDGE:

Height of the Retaining Wall	=	20 feet
Slope Angle of Retained Slope	=	0 degree
Dip Angle of Critical Wedge	=	56 degree

SHEAR STRENGTH PARAMETERS:

Unit Weight	=	135 pcf
Cohesion	=	90 psf
Friction Angle	=	30.5 degree
Mobilized Cohesion	=	60.0 psf
Mobilized Friction Angle	=	21.5 degree

REQUIRED FACTOR OF SAFETY = 1.5

RESULTS

Dip Angle of Critical Slip Surface	=	56 degree
Total Weight of Active Wedge	=	18212 lbs
Frictional Resistance (Cm * L)	=	1447 lbs
Required External Force for Wall	=	10909 lbs
Required Equivalent Fluid Pressure	=	54.5 psf/ft

** Rankine Wedge is not the most critical wedge **

RECOMMENDED EFP VALUE:

Triangular-Distributed EFP = 54.5x(1+sin(30.5))=82psf/ft

Trapezoidal-Distributed LF = EFP(Tri)/1.6
= 52H psf/ft

Figure 10

Equivalent Fluid Pressure (Free Body Diagram Method)

Program Made by C. Y. Geotech, Inc.

Project Name:
CYG-07-4948 30 feet Retaining Wall / Level Backfill (Soil)

GEOMETRY OF CRITICAL ACTIVE WEDGE:

Height of the Retaining Wall	=	30 feet
Slope Angle of Retained Slope	=	0 degree
Dip Angle of Critical Wedge	=	56 degree

SHEAR STRENGTH PARAMETERS:

Unit Weight	=	135 pcf
Cohesion	=	90 psf
Friction Angle	=	30.5 degree
Mobilized Cohesion	=	60.0 psf
Mobilized Friction Angle	=	21.5 degree

REQUIRED FACTOR OF SAFETY = 1.5

RESULTS

Dip Angle of Critical Slip Surface	=	56 degree
Total Weight of Active Wedge	=	40976 lbs
Frictional Resistance (Cm * L)	=	2171 lbs
Required External Force for Wall	=	25772 lbs
Required Equivalent Fluid Pressure	=	57.3 psf/ft

** Rankine Wedge is not the most critical wedge **

RECOMMENDED EFP VALUE:

Triangular-Distributed EFP = 57.3x(1+sin(30.5))=87psf/ft

Trapezoidal-Distributed LF = EFP(Tri)/1.6
= 55H psf/ft

Figure 11

Wall Height (ft)	Triangular-Distributed Equivalent Fluid Pressure, psf/ft	Trapezoidal-Distributed Lateral Force psf per linear foot of width
10	70 + Surcharge	44H + Surcharge
20	82 + Surcharge	52H + Surcharge
30	87 + Surcharge	55H + Surcharge

H: retaining height of subterraneous building wall

Four (4) additional wedge slope stability analyses using the Free Body Diagram method were performed to evaluate the stability of 10-foot, 20-foot, 30-foot and 35-foot high temporary excavations in alluvial soil. The peak shear strength parameters of alluvial soil were used in analyses. The results of analyses are presented in Figures 12, 13, 14 and 15. The analyses indicated factors of safety less than the minimum code requirement for all four cases. The lateral forces and equivalent fluid pressures in the following table can be used in the design of shoring system.

Height of Temporary Excavation, (ft)	10	20	30	35
Equivalent Fluid Pressure, psf/ft	20 + Surcharge	34 + Surcharge	39 + Surcharge	40 + Surcharge

9.0 CONCLUSIONS AND RECOMMENDATIONS

Based upon the findings of this investigation, the development of the proposed mixed use buildings at the subject site is feasible from a geotechnical engineering viewpoint provided the recommendations of this report are incorporated into design and implemented during construction.

Conventional spread footings, mat foundation or deep foundation such as skin friction piles entirely founded into competent alluvium can be used to support the proposed mixed use buildings.

Shoring system will be required for deep temporary excavations. Soldier piles with tieback-lagging system can be used for shoring of temporary excavations. A monitoring program for lateral displacement of soldier piles will be required for the shoring system.

Provided the recommendations in this report are properly incorporated into design and implemented during construction, the proposed mixed use building will be safe from geologic hazards including settlement, landsliding, slippage and liquefaction and the development of the proposed mixed condominium/retailer buildings will not adversely affect the geologic stability of the site and adjacent properties.

9.1 Site Preparation

In the areas of exterior concrete slab-on-grade, asphalt pavement and interlocking paver, the existing soil should be removed to a depth of 4 feet below the existing grade or 2 feet below the bottoms of concrete slabs, pavements or pavers, whichever is deeper, and then recompacted to be compacted fill for the support of concrete slabs, pavements and pavers. The removal and recompaction should be extended a minimum of 4 feet beyond the boundaries of concrete slabs, pavements and pavers in all directions. If any soft spot is encountered on the bottom of removal, the soft spot should be over-excavated to underlying competent soil under the direction of CYG prior to the placing fill soil. The removal and recompaction can be limited to property lines. The bottom of removal should be inspected and approved by the representative of CYG and th City Grading Inspector prior to the placing of fill soil.

Equivalent Fluid Pressure (Free Body Diagram Method)

Program Made by C. Y. Geotech, Inc.

Project Name:
CYG-07-4948 10 feet Temporary Cut / Level Backfill (Soil)

GEOMETRY OF CRITICAL ACTIVE WEDGE:

Height of the Retaining Wall	=	10 feet
Slope Angle of Retained Slope	=	0 degree
Dip Angle of Critical Wedge	=	58 degree

SHEAR STRENGTH PARAMETERS:

Unit Weight	=	120 pcf
Cohesion	=	140 psf
Friction Angle	=	31 degree
Mobilized Cohesion	=	112.0 psf
Mobilized Friction Angle	=	25.7 degree

REQUIRED FACTOR OF SAFETY = 1.25

RESULTS

Dip Angle of Critical Slip Surface	=	58 degree
Total Weight of Active Wedge	=	3749 lbs
Frictional Resistance (Cm * L)	=	1321 lbs
Required External Force for Wall	=	964 lbs
Required Equivalent Fluid Pressure	=	19.3 psf/ft

** Rankine Wedge is not the most critical wedge **

RECOMMENDED EFP VALUE:

Triangular-Distributed EFP = 20 psf/ft

Figure 12

Equivalent Fluid Pressure (Free Body Diagram Method)

Program Made by C. Y. Geotech, Inc.

Project Name:
CYG-07-4948 20 feet Temporary Cut / Level Backfill (Soil)

GEOMETRY OF CRITICAL ACTIVE WEDGE:

Height of the Retaining Wall	=	20 feet
Slope Angle of Retained Slope	=	0 degree
Dip Angle of Critical Wedge	=	58 degree

SHEAR STRENGTH PARAMETERS:

Unit Weight	=	120 pcf
Cohesion	=	140 psf
Friction Angle	=	31 degree
Mobilized Cohesion	=	112.0 psf
Mobilized Friction Angle	=	25.7 degree

REQUIRED FACTOR OF SAFETY = 1.25

RESULTS

Dip Angle of Critical Slip Surface	=	58 degree
Total Weight of Active Wedge	=	14997 lbs
Frictional Resistance (Cm * L)	=	2641 lbs
Required External Force for Wall	=	6673 lbs
Required Equivalent Fluid Pressure	=	33.4 psf/ft

** Rankine Wedge is not the most critical wedge **

RECOMMENDED EFP VALUE:

Triangular-Distributed EFP = 34 psf/ft

Figure 13

Equivalent Fluid Pressure (Free Body Diagram Method)

Program Made by C. Y. Geotech, Inc.

Project Name:
CYG-07-4948 30 feet Temporary Cut / Level Backfill (Soil)

GEOMETRY OF CRITICAL ACTIVE WEDGE:

Height of the Retaining Wall	=	30 feet
Slope Angle of Retained Slope	=	0 degree
Dip Angle of Critical Wedge	=	58 degree

SHEAR STRENGTH PARAMETERS:

Unit Weight	=	120 pcf
Cohesion	=	140 psf
Friction Angle	=	31 degree
Mobilized Cohesion	=	112.0 psf
Mobilized Friction Angle	=	25.7 degree

REQUIRED FACTOR OF SAFETY = 1.25

RESULTS

Dip Angle of Critical Slip Surface	=	58 degree
Total Weight of Active Wedge	=	33743 lbs
Frictional Resistance (Cm * L)	=	3962 lbs
Required External Force for Wall	=	17128 lbs
Required Equivalent Fluid Pressure	=	38.1 psf/ft

** Rankine Wedge is not the most critical wedge **

RECOMMENDED EFP VALUE:

Triangular-Distributed EFP = 39 psf/ft

Figure 14

Equivalent Fluid Pressure (Free Body Diagram Method)

Program Made by C. Y. Geotech, Inc.

Project Name:
CYG-07-4948 35 feet Temporary Cut / Level Backfill (Soil)

GEOMETRY OF CRITICAL ACTIVE WEDGE:

Height of the Retaining Wall	=	35 feet
Slope Angle of Retained Slope	=	0 degree
Dip Angle of Critical Wedge	=	58 degree

SHEAR STRENGTH PARAMETERS:

Unit Weight	=	120 pcf
Cohesion	=	140 psf
Friction Angle	=	31 degree
Mobilized Cohesion	=	112.0 psf
Mobilized Friction Angle	=	25.7 degree

REQUIRED FACTOR OF SAFETY = 1.25

RESULTS

Dip Angle of Critical Slip Surface	=	58 degree
Total Weight of Active Wedge	=	45928 lbs
Frictional Resistance (Cm * L)	=	4622 lbs
Required External Force for Wall	=	24134 lbs
Required Equivalent Fluid Pressure	=	39.4 psf/ft

** Rankine Wedge is not the most critical wedge **

RECOMMENDED EFP VALUE:

Triangular-Distributed EFP = 40 psf/ft

Figure 15

9.2 Conventional Spread Footing

Conventional spread footings founded into competent alluvium can be used to support of the proposed mixed use building. The following recommendations can be used in the design of conventional spread footings supported by alluvium.

a. Conventional spread footings should be entirely supported by competent alluvium.

b. Continuous spread footings should have a minimum width of 12 inches and a minimum embedment depth of 24 inches into competent alluvium.

c. Isolated footings should have a minimum width of 24 inches and a minimum embedment depth of 24 inches into competent alluvium.

d. An allowable vertical bearing pressure of 3000 pounds per square foot (psf), including dead and frequently applied live loads, may be used in the design of footings with minimum footing width and embedment depth. The bearing capacity can be increased by 500 psf for each additional foot of footing width or embedment depth, to a maximum bearing capacity of 6000 psf. The vertical bearing capacity can be increased by one-third when considering short duration wind and seismic loads.

e. For areas with no subterraneous parking levels, an allowable vertical bearing pressure of 1500 pounds per square foot (psf), including dead and frequently applied live loads, may be used in the design of footings with minimum footing width and embedment depth. The bearing capacity can be increased by 300 psf for each additional foot of footing width or embedment depth, to a maximum bearing capacity of 3000 psf. The vertical bearing capacity can be increased by one-third when considering short duration wind and seismic loads.

f. Lateral force can be resisted by frictional resistance and passive earth pressure. An allowable friction coefficient of 0.4 and an allowable passive earth pressure of 400 psf/ft, to a maximum of 4000 psf, can be used to resist lateral loads. When combined passive earth pressure and frictional resistance, the passive earth pressure component should be reduced by one-third.

g. All footings should have a minimum reinforcement of two No.4 steel bars near the top and two No.4 steel bars near the bottom. Where footing excavation and stem wall height exceeds a combined depth of 3 feet, one No.4 steel bar should be placed vertically every 3 feet. These parameters should be reviewed by the Project Structural Engineer and revised as required to accommodate intended use.

h. Prior to the placement of steel in footing excavations, an inspection should be made by the representative of CYG and the City Grading Inspector to ensure that footing excavations expose competent alluvium and are free of loose and disturbed soils.

9.3 Mat Foundation

A mat foundation is a large concrete slab which transmits the loading from several columns in a building or the entire building loads to the ground. A mat foundation is often used when the soil is of such poor quality, or the column loads are so large, that more than 50% of the building-plan area is covered by footings. The mat foundation differs from an individual footing. Mat foundation will require much thicker concrete slab and negative reinforcing steel.

The following recommendations can be used in the design of mat foundation.

a. Mat foundation should be designed by the Project Structural Engineer.

b. Mat foundation should be entirely supported by alluvial soil

c. A subgrade reaction coefficient of 300 kcf can be used in the design of mat foundation.

d. An allowable vertical bearing pressure of 3000 psf, including dead and frequently applied live loads, can be used in the design of mat foundation. The allowable vertical bearing capacities can be increased by one-third when considering short duration wind or seismic loads.

e. Lateral force can be resisted by frictional resistance and passive earth pressure. An allowable friction coefficient of 0.4 and an allowable passive earth pressure of 400 psf/ft, to a maximum of 2000 psf, can be used to resist lateral loads. When combining passive earth pressure and frictional resistance, the passive earth pressure component should be reduced by one-third.

f. Prior to the placement of steel in footing trenches, an inspection should be made by the representative of CYG and the City Grading Inspector to ensure that the footing trenches exposes competent alluvial soil.

<u>9.4 Skin Friction Pile</u>

Alternatively, skin friction piles supported by competent alluvium can also be used to support the proposed mixed use building. The following recommendations can be used in the design of skin friction piles.

a. Skin friction piles can be designed as cast-in-place piles entirely supported by competent alluvium.

b. Piles can be assumed fixed at 2 feet below the lowest adjacent grade. The piles should be tied with grade beams in a minimum of two directions for building structures.

c. Skin friction piles should be embedded a minimum of 15 feet into competent alluvium but not less than the depth required for adequate vertical support and lateral resistance.

d. An allowable skin friction of 500 psf can be used to determine the minimum pile length required for vertical support. The earth materials above the fixity point should be assumed providing no vertical support. The allowable skin friction can be increased by 1/3 when considering short duration wind or seismic loads.

e. Allowable passive earth pressure may be computed as an equivalent fluid having a density of 400 pcf, to a maximum passive earth pressure of 6000 psf. The earth material above the fixity point should be assumed providing no lateral support. The passive earth pressure may be increased by 100% for isolated piles. Piles with spacing greater than 3 times of pile diameter can be considered as isolated piles.

f. Drilling of pile holes should be observed and approved by the representative of CYG and the City Grading Inspector prior to placing steel. If clay cake occurs on the side wall, the clay cake should be clean and removed from drilled holes prior to placing steel.

<u>9.5 Subterraneous Basement Wall</u>

The following recommendations can be used in the design of the subterraneous basement walls for the proposed parking levels.

a. Conventional spread footings, mat foundation and skin friction piles founded into competent alluvium as recommended in Sections 9.2, 9.3 and 9.4 can be used to support the subterraneous basement walls.

b. The triangular-distributed equivalent fluid pressures and trapezoidal-distributed lateral forces listed in the following table can be used the design of the subterraneous basement walls. Any anticipated superimposed loading within a 1:1 plane projected upward from the wall bottom, except retained soil, should be considered as surcharge and provided for in the design.

Wall Height (ft)	Triangular-Distributed Equivalent Fluid Pressure, psf/ft	Trapezoidal-Distributed Lateral Force psf per linear foot of width
10	70 + Surcharge	44H + Surcharge
20	82 + Surcharge	52H + Surcharge
30	87 + Surcharge	55H + Surcharge

H: retaining height of subterraneous building wall

Subterraneous basement walls are usually designed as restrained retaining walls using trapezoidal-distributed lateral forces. Illustration of the triangular-distributed equivalent fluid pressures and trapezoidal-distributed lateral forces are shown on Figure 16.

c. Subterraneous basement wall should be provided with a minimum of one 4-inch diameter perforated PVC pipe in a gravel envelope behind the bottom of retaining wall. One-foot thick zone of clean, granular, free-draining soil or gravel should be placed behind the wall to 2 feet below of the finished grade. Compacted fill with less permeable soil should be placed for the upper 2 feet of wall backfill.

d. Retaining wall backfill must be compacted to a minimum dry density of 90% of the maximum dry density as determined by ASTM Standard D-1557-02. Fill soil should be compacted to a minimum of 95% of the maximum dry density if clay content of the fill soil is less than 15%.

e. The subdrain system should be inspected and approved by the representative of CYG and the City Grading Inspector prior to placing additional gravel or compacted fill above the subdrain system.

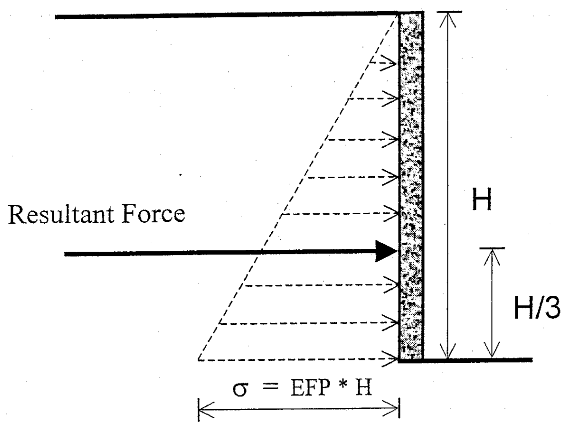
f. The compacted fill for retaining wall backfill should be tested by the representative of CYG. If the test indicated the tested layer has a dry density test less than the minimum requirement, the tested layer should be removed, recompacted and retested until the required compaction degree is achieved.

g. The subterraneous basement wall should be provided with a proper waterproofing system to prevent the migration of subsurface water to the interior side of the wall. The waterproofing system should be designed by the Project Civil Engineer or Structural Engineer.

9.6 Waterproofing

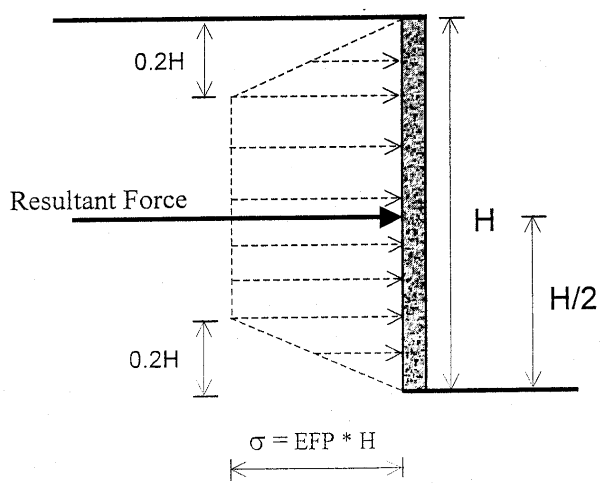
Retaining walls are susceptible to moisture penetration and therefore, a waterproofing system on the backside of the basement walls can be installed to provide the walls with a protection against moisture penetration. No waterproofing system can guarantee 100% protection. Conventional waterproofing materials, such as asphalt emulsion, have often proved ineffective. Waterproofing paints applied to the exterior side of walls are more effective than conventional asphalt emulsion. Bentonitic clay panels have also proven to be very effective.

Nonrestrained Wall



Resultant Force = $\frac{1}{2} \times \sigma \times H = \frac{1}{2} \times EFP \times H^2$

Restrained Wall



Resultant Force = $0.8 \times \sigma \times H = 0.8 \times EFP \times H^2$

C. Y. GEOTECH, INC. Engineering Geology and Geotechnical Engineering	**Triangular and Trapezoidal Distribution Diagram of Lateral Force**	
	CYG-03-4948	**Figure 16**

It is recommended that foundation contractor or retaining wall contractor provide recommendations for proven waterproofing systems to be utilized. The waterproofing system should be evaluated and approved by the Project Civil Engineer and/or the Project Structural Engineer who designs the retaining wall.

9.7 Settlement

The total and differential settlements of the proposed mixed use buildings supported by conventional spread footings, mat foundation and/or skin friction pile as recommended are anticipated to be within tolerable limits. Total settlements of the footings are expected to be less than 1 inch. Differential settlement should be less than ½ inch in a span of 20 feet.

Due tot he high structural loads, It is recommended that a detailed foundation settlement be performed to evaluate the total and differential settlements when the preliminary design of structural loads and footing patterns are available.

It should be noted that the evaluation of settlement is based on the assumption that the entire area of the proposed mixed use buildings and its surrounding areas will be provided with adequate surface and subsurface drainage devices and that the drainage systems will be properly and constantly maintained. Additional settlement caused by local bearing failure or soil lubrication may occur if foundation soil is saturated o nearly saturated. In order to avoid the migration of a significant amount of surface and subsurface water to the bearing soil of the proposed building structures, the recommendations in the section of “Drainage Control” should be incorporated into the design and implemented during construction.

9.8 Slab-On-Grade

Interior concrete slabs-on-grade should be entirely supported by competent alluvium. Exterior concrete slabs-on-grade should be either entirely supported by compacted fill or entirely supported by competent alluvium which is approximately 4 feet below the existing ground surface. If exterior concrete slabs-on-grade supported by compacted fill are proposed, the existing soil in the exterior concrete slab areas should be removed to a minimum of 4 feet below the existing ground or 2 feet foot below the bottom of the concrete slabs, whichever is deeper, and then recompacted to be compacted fill for slab support. The removal and recompaction should be extended horizontally a minimum of 4 feet beyond the boundaries of exterior concrete slabs. Crack control joints should be created in exterior slabs and walkways.

Concrete slabs-on-grade for parking levels should be designed for a minimum thickness of 5 inches, reinforced with No.4 bars at 12 inches on centers, both ways. Concrete slab-on-grade for non-parking levels should be designed for a minimum thickness of 4 inches, reinforced with No.4 bars at 16 inches on centers, both ways. Reinforcement should be properly supported to assure desired mid-height placement. A 10-mil plastic vapor barrier should be placed below the floor slabs in moisture sensitive areas. The vapor barrier should be sandwiched by two 2-inch sand layers to protect the vapor barrier from punctures and to aid in the concrete cure. These parameters should be reviewed by the Project Structural Engineer and revised as required to accommodate intended use.

Decking, slabs and walkways are likely to experience cracking as the results of the curing process of the concrete. Shrinkage cracks are very difficult to prevent from occurring. Expansion joints are commonly installed within exterior decks in an effort to control the location of the inevitable cracks. Interior slabs however are typically not provided with expansion joint, making cracking more random. The recommended steel reinforcement is intended to reduce the severity of cracking and must be properly installed to ensure proper performance.

Rigid or brittle floor covering, such as tile or marble may also experience cracking during the curing process of the concrete slab underneath and/or minor settlement. Providing a slip sheet between the slab and floor covering will help to reduce cracking of the floor covering.

9.9 Asphalt Concrete Pavement and Interlocking Pavers

Asphalt concrete pavement and interlocking paver should be either entirely supported by compacted fill or entirely supported by competent alluvium which is approximately 4 feet below the existing ground surface. If asphalt concrete pavement and interlocking paver supported by compacted fill are proposed, the existing soil in the areas of asphalt concrete pavement and interlocking paver should be removed to a minimum of 4 feet below the existing ground or 2 feet foot below the bottom of the asphalt concrete pavement and interlocking paver, whichever is deeper, and then recompacted to be compacted fill. The removal and recompaction should be extended horizontally a minimum of 4 feet beyond the boundaries of asphalt concrete pavement and interlocking paver.

The recommendations in sections 9.9.1 and 9.9.2 can be used in the design of asphalt concrete pavements and interlocking pavers, respectively.

9.9.1 Asphalt Concrete Pavement

The calculations of structural sections for asphalt concrete pavement were performed based on the method introduced in ”Flexible Pavement Structural Section Design Guide for California Cities and Counties,” third edition, January 1979. A R-Value of 25 and traffic indexes of 4, 5 and 6 were assumed in th calculations of structural section. The results of calculations are shown on Figures 17, 18 and 19 and summarized in the following Table. A traffic index of 4 is recommended for passenger car area. A traffic index of 5 is recommended for delivery of working zones and light truck areas. A traffic index 6 is recommended for dock areas and heavy truck areas.

Traffic Index (TI)	4	5	6
Thickness of Asphalt Concrete, inch	3	4	5
Thickness of Aggregate Base, inch	4	4	6

Compaction tests will be required for the aggregate base. A minimum relative compaction of 95% is required for the aggregate base. If the asphalt and block concrete pavements are to be supported by compacted fill, the fill soil should be compacted to a minimum of 90% of the maximum dry density per ASTM D-1557-02. Fill soil should be compacted to a minimum of 95% of the maximum dry density if clay content of the fill soil is less than 15%.

9.9.2 Interlocking Concrete Paver

Interlocking concrete pavers should be supported by a minimum of 1 inch of sand layer overlying a minimum of 4 inches of aggregate base. A layer of soil separation fabrics is recommended be placed between the sand layer and the underlying aggregate base. A typical section for block concrete pavement is shown on Figure 20. The sand should be clean sand confirming the grading requirement of ASTM C33.

The interlocking concrete pavers for block concrete pavement should have a minimum thickness of 60 millimeters (mm). A minimum of 80 mm may be considered for areas where may have the activity of two-axle trucks. The joints between pavers can be filled with sand or slurry. The sand layer and pavers should be held tightly together by edge restraints such as submerged concrete edge or steel edge.

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*********************************************************
* FLEXIBLE PAVEMENT DESIGN USING CALIFORNIA DESIGN GUIDE *
*********************************************************

PROJECT NAME : Salazar / Victory / TI = 4

TRAFFIC INDEX = 4

R-VALUES :
  AGGREGATE BASE     = 78
  AGGREGATE SUBBASE  = 25
  BASEMENT MATERIAL  = 25

GRAVEL EQUIVALENT :
  AGGREGATE BASE     = 3.4 INCHES
  AGGREGATE SUBBASE  = 11.5 INCHES
  BASEMENT MATERIAL  = 11.5 INCHES

GRAVEL EQUIVALENT FACTOR :
  AGGREGATE BASE     = 2.83
  AGGREGATE SUBBASE  = 1.1
  BASEMENT MATERIAL  = 1

MINIMUM THICKNESS DESIGN :
   ASPHALT CONCRETE   = 1.2 INCHES
   AGGREGATE BASE     = 7.4 INCHES
   AGGREGATE SUBBASE  = 0 INCHES

FULL DEPTH ASPHALT CONCRETE DESIGN :
   ASPHALT CONCRETE   = 4.1 INCHES


********************************
*   PROPOSED PAVEMENT DESIGN    *
********************************

DESIGN THICKNESS OF ASPHALT CONCRETE   = 3 INCHES
DESIGN THICKNESS OF AGGREGATE BASE     = 4 INCHES
DESIGN THICKNESS OF AGGREGATE SUBBASE  = 0 INCHES

TOTAL GRAVEL EQUIVALENT REQUIRED = 11.5 INCHES
DESIGN TOTAL GRAVEL EQUIVALENT   = 12.9 INCHES

DESIGN GRAVEL EQUIVALENT > REQUIRED GRAVEL EQUIVALENT
```

C. Y. GEOTECH, INC. Engineering Geology and Geotechnical Engineering	Pavement Design Calculation Traffic Index = 4
CYG-03-4948	Figure 17

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*********************************************************
* FLEXIBLE PAVEMENT DESIGN USING CALIFORNIA DESIGN GUIDE *
*********************************************************

PROJECT NAME : Salazar / Victory / TI = 5

TRAFFIC INDEX = 5

R-VALUES :
  AGGREGATE BASE     = 78
  AGGREGATE SUBBASE  = 25
  BASEMENT MATERIAL  = 25

GRAVEL EQUIVALENT :
  AGGREGATE BASE     = 4.2 INCHES
  AGGREGATE SUBBASE  = 14.4 INCHES
  BASEMENT MATERIAL  = 14.4 INCHES

GRAVEL EQUIVALENT FACTOR :
  AGGREGATE BASE     = 2.53
  AGGREGATE SUBBASE  = 1.1
  BASEMENT MATERIAL  = 1

MINIMUM THICKNESS DESIGN :
   ASPHALT CONCRETE   = 1.7 INCHES
   AGGREGATE BASE     = 9.2 INCHES
   AGGREGATE SUBBASE  = 0 INCHES

FULL DEPTH ASPHALT CONCRETE DESIGN :
   ASPHALT CONCRETE   = 5.7 INCHES


********************************
*   PROPOSED PAVEMENT DESIGN   *
********************************

DESIGN THICKNESS OF ASPHALT CONCRETE   = 4 INCHES
DESIGN THICKNESS OF AGGREGATE BASE     = 4 INCHES
DESIGN THICKNESS OF AGGREGATE SUBBASE  = 0 INCHES

TOTAL GRAVEL EQUIVALENT REQUIRED = 14.4 INCHES
DESIGN TOTAL GRAVEL EQUIVALENT   = 14.5 INCHES

DESIGN GRAVEL EQUIVALENT > REQUIRED GRAVEL EQUIVALENT
```
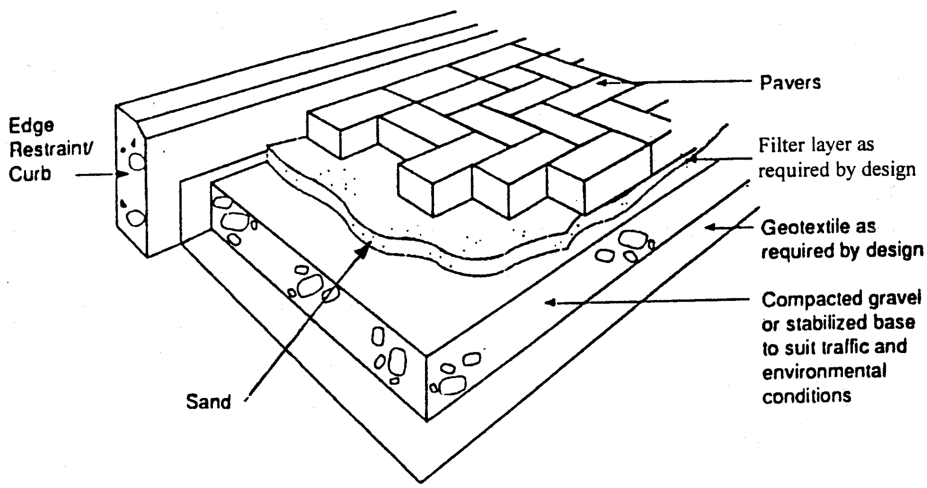

C. Y. GEOTECH. INC. Engineering Geology and Geotechnical Engineering	Pavement Design Calculation Traffic Index = 5	
	CYG-03-4948	Figure 18

```
DESIGN GRAVEL EQUIVALENT > REQUIRED GRAVEL EQUIVALENT
**********************************************************
* FLEXIBLE PAVEMENT DESIGN USING CALIFORNIA DESIGN GUIDE *
**********************************************************

PROJECT NAME : Salazar / Victory / TI = 6

TRAFFIC INDEX = 6

R-VALUES :
  AGGREGATE BASE     = 78
  AGGREGATE SUBBASE  = 25
  BASEMENT MATERIAL  = 25

GRAVEL EQUIVALENT :
  AGGREGATE BASE     = 5.1 INCHES
  AGGREGATE SUBBASE  = 17.3 INCHES
  BASEMENT MATERIAL  = 17.3 INCHES

GRAVEL EQUIVALENT FACTOR :
  AGGREGATE BASE     = 2.31
  AGGREGATE SUBBASE  = 1.1
  BASEMENT MATERIAL  = 1

MINIMUM THICKNESS DESIGN :
   ASPHALT CONCRETE   = 2.2 INCHES
   AGGREGATE BASE     = 11.1 INCHES
   AGGREGATE SUBBASE  = 0 INCHES

FULL DEPTH ASPHALT CONCRETE DESIGN :
   ASPHALT CONCRETE   = 7.5 INCHES


********************************
*   PROPOSED PAVEMENT DESIGN   *
********************************

DESIGN THICKNESS OF ASPHALT CONCRETE   = 5 INCHES
DESIGN THICKNESS OF AGGREGATE BASE     = 6 INCHES
DESIGN THICKNESS OF AGGREGATE SUBBASE  = 0 INCHES

TOTAL GRAVEL EQUIVALENT REQUIRED = 17.3 INCHES
DESIGN TOTAL GRAVEL EQUIVALENT   = 18.2 INCHES

DESIGN GRAVEL EQUIVALENT > REQUIRED GRAVEL EQUIVALENT
```

C. Y. GEOTECH. INC. Engineering Geology and Geotechnical Engineering	Pavement Design Calculation Traffic Index = 6	
	CYG-03-4948	Figure 19



Typical Components of an Interlocking Concrete Pavement

C.Y. GEOTECH, INC. Engineering Geology and Geotechnical Engineering	**Typical Section for Block Concrete Pavement** **CYG-03-4948** — **Figure 20**

Compaction tests will be required for the aggregate base. A minimum relative compaction of 95% is required for the aggregate base. If interlocking concrete pavers are to be supported by compacted fill, the fill soil should be compacted to a minimum of 90% of the maximum dry density per ASTM D-1557-02. Fill soil should be compacted to a minimum of 95% of the maximum dry density if clay content of the fill soil is less than 15%.

9.10 Fill Placement and Soil Compaction

Fill placement and soil compaction will be required for retaining wall backfill. Fill placement and soil compaction will be required if concrete slabs, asphalt concrete pavement and/or interlocking pavers supported by compacted fill are proposed. All grading work and fill placement should be performed in conformance with the current grading ordinances of the City of Los Angeles. The following general guidelines can be used as a basis for quality control of fill placement and soil compaction.

a. Remove vegetation, loose soil and all other deleterious materials in the fill placement area prior to fill placement.

b. The bottom of removal should be scarified a minimum of 6 inches, thoroughly moistened and mixed to near the optimum moisture content and then properly compacted prior to placing fill soil.

c. The bottom to receive fill soil should be inspected and approved by the representative of CYG and the City Grading Inspector prior to placing any fill soil. If loose spot is observed on the bottom to receive fill soil, the soil in the loose spot should be overexcavated to the underlying competent soil under the observation of CYG and then recompacted to be compacted fill prior to placing fill soil in other fill placement areas.

d. The excavated onsite soils, cleaned of deleterious material, can be used for compacted fill. Rock larger than 6 inches in the longest side should not be buried or placed in compacted fill.

e. Compacted fill should be placed in controlled layers which, when compacted, should not exceed 6 inches in thickness.

f. All compacted fill should be thoroughly moistened and mixed to near the optimum moisture content and then compacted to a minimum dry density 90% of the maximum dry density, as determined by ASTM Standard D-1557-02. Fill soil should be compacted to a minimum of 95% of the maximum dry density if clay content of the fill soil is less than 15%.

g. At least one field density test should be made for every 2 feet of vertical lift. Both sand cone method and nuclear gauge method will be required for field density tests. If the test indicates a dry density less than th required compaction degree, the tested layer should be removed, recompacted and retested until the required compaction degree is achieved.

h. All fill placement and soil compaction should be observed and tested by the representative of CYG. The excavated bottom to receive fill soil should be observed by the representative of CYG and the City Grading Inspector prior to placing any fill soil.

It should be noted that it is the responsibility of you, your representative or your contractor to perform the required fill placement and soil compaction, and to notify CYG to perform the required inspection and density testing.

All lines and grades for the proposed development should be provided by you, the general contractor or the grader. CYG should not be assumed any responsibility for lines and grades.

A soil compaction report will be required for the fill placement and soil compaction. A soil compaction report with a certificate for compacted fill should be submitted to the City of Los Angeles after the completion of fill placement and soil compaction.

9.11 Temporary Excavation

Temporary excavations to a maximum depth of approximately 30 to 35 feet in vertical will be required for the construction of the subterraneous parking levels. The slope stability analyses indicated that shoring protection is required for temporary excavations more than 5 feet in depth.

Temporary excavation below the 1:1 lines projected downward from the bottom of adjacent structures or footings will be considered the removal of vertical and lateral support from the adjacent structures or footings. If temporary excavation removes vertical or lateral support of any adjacent structure or footings, the temporary excavation should be protected by a shoring system or be conducted using the A/B/C slot cutting method.

Based on the findings of slope stability analyses and the evaluations of temporary excavations required for the subject project, the general recommendations in the following table can be used in preliminary design of temporary excavations.

Height of Excavation (H), ft	Necessity of Shoring	Equivalent Fluid Pressure, psf/ft	Remarks
$5 < H \leq 10$	Shoring	20	see sections 9.11.1 to 9.11.3
$10 < H \leq 20$	Shoring	34	see sections 9.11.1 to 9.11.3
$20 < H \leq 30$	Shoring	39	see sections 9.11.1 to 9.11.3
$30 < H \leq 35$	Shoring	40	see sections 9.11.1 to 9.11.3
$H > 35$	Shoring	----	additional evaluations are required

9.11.1 Soldier Piles

Soldier piles can be used as a shoring system to protect temporary excavations. The equivalent fluid pressures in the following table can be used in the design of soldier piles. The equivalent fluid pressures for soldier piles can be shared by tie-back anchors if soldier piles are to be deigned as tie-back soldier piles.

Height of Temporary Excavation, (ft)	$5 < H \leq 10$	$10 < H \leq 20$	$20 < H \leq 30$	$30 < H \leq 35$
Equivalent Fluid Pressure, psf/ft	20 + Surcharge	34 + Surcharge	39 + Surcharge	40 + Surcharge

Soldier piles should be embedded a minimum of 10 feet below the lowest adjacent grade of competent alluvium but not less than the depth required for adequate lateral resistance. The pile spacing should not be greater than 10 feet. Piles can be assumed fixed at 3 feet below the lowest adjacent grade of competent alluvium.

Passive earth pressure provided by competent alluvium can be computed as an equivalent fluid having a density of 400 pcf, to a maximum passive earth pressure of 6000 psf. The soils above the fixity point should be assumed providing no lateral support. The passive earth pressure may be increased by 100% for isolated piles. Piles with spacing greater than 3 times of the pile diameter can be considered as isolated piles. Drilling of pile holes should be observed and approved by the representative of CYG and the City Grading Inspector prior to placing steel.

9.11.2 Tie-Back System

Tie-back anchors can be used to share the stabilization fore recommended for soldier piles. The anchorage of the tie-back system should be assumed starting from a plane which is 60 degrees projected upward from the toe of temporary excavation. The skin frictions listed in the following table can be used to calculate the resisting force provided by the anchorages of tie-back anchors. The calculations of skin friction for tie-back anchorage are shown on Figure 21.

Depth to Tie-Back Anchorage, ft	10 - 15	15- 20	20 - 25	25 - 30	30 - 35	35 - 40	> 40
Anchorage Resisting Strength, psf	430	610	790	970	1150	1330	1510

* Depth to Tie-back Anchorage = Depth to Top of Tieback Anchor + Total Length of Tie-back Anchor x Sin(inclination angle)

All tie-back anchors should be pre-tested to at least 150% of the design load. At least 5% of the tie-back anchors should be pre-tested to 200% of the design load. The tests should be performed by the shoring contractor. The installation of the tie-back anchors should be inspected and approved by the representative of CYG.

The following recommendations should be followed in the 150% design load tests:

1. The 150% design load tests should be tested for each tie-back anchor. The test load should be applied to each anchor by a hydraulic jack, calibrated to 1% accuracy by a licensed testing laboratory.

2. Fifteen minute after the application of the initial load, the test load should be checked and recorded. The test load should be brought back to 150% of design load if there is a drop in the gage reading. The elongation of the anchor should be recorded (e_1).

3. Check the gage reading at 30 minute intervals for the next two hours after the initial 15 minute reading. The test load should be brought back to 150% of design load if there is a drop in the gage reading. The elongation of the anchor should be recorded at the end of two hours (e_2).

4. An anchor is considered to have passed the 150% load test if the difference of the elongation between e_1 and e_2 is equal to or less than 0.25 inches.

The following recommendations should be followed in the 200% design load tests:

1. The 200% design load tests should be tested for each tie-back anchor. The test load should be applied to each anchor by a hydraulic jack, calibrated to 1% accuracy by a licensed testing laboratory.

2. Fifteen minute after the application of the initial load, the test load should be checked and recorded. The test load should be brought back to 200% of design load if there is a drop in the gage reading. The elongation of the anchor should be recorded (e_1).

CALCULATION OF SKIN FRICTION OF TIE-BACK ANCHORAGE

Unit Weight (γ)	=	120 pcf
Cohesion (c)	=	140 psf
Friction Angle (ϕ)	=	31 deg

When Depth of Tie-Back Anchorage = 5 feet
Skin Friction = 140 + 120 x 5 x Tan(31) = 500 psf
Allowable Skin Friction = 500 / 2 = 250 Says 250 psf

When Depth of Tie-Back Anchorage = 10 feet
Skin Friction = 140 + 120 x 10 x Tan(31) = 861 psf
Allowable Skin Friction = 861 / 2 = 430 Says 430 psf

When Depth of Tie-Back Anchorage = 15 feet
Skin Friction = 140 + 120 x 15 x Tan(31) = 1222 psf
Allowable Skin Friction = 1222 / 2 = 611 Says 610 psf

When Depth of Tie-Back Anchorage = 20 feet
Skin Friction = 140 + 120 x 20 x Tan(31) = 1582 psf
Allowable Skin Friction = 1582 / 2 = 791 Says 790 psf

When Depth of Tie-Back Anchorage = 25 feet
Skin Friction = 140 + 120 x 25 x Tan(31) = 1942 psf
Allowable Skin Friction = 1942 / 2 = 971 Says 970 psf

When Depth of Tie-Back Anchorage = 30 feet
Skin Friction = 140 + 120 x 30 x Tan(31) = 2303 psf
Allowable Skin Friction = 2302 / 2 = 1151 Says 1150 psf

When Depth of Tie-Back Anchorage = 35 feet
Skin Friction = 140 + 120 x 35 x Tan(31) = 2664 psf
Allowable Skin Friction = 2664 / 2 = 1332 Says 1330 psf

When Depth of Tie-Back Anchorage = 40 feet
Skin Friction = 140 + 120 x 40 x Tan(31) = 3024 psf
Allowable Skin Friction = 3024 / 2 = 1512 Says 1510 psf

Figure 21

3. Check the gage reading at two 2) hour intervals for24 hours after the initial 15 minute reading. The test load should be brought back to 200% of design load if there is a drop in the gage reading. The elongation of the anchor should be recorded at the end of two hours (e_2).

4. An anchor is considered to have passed the 200% load test if the difference of the elongation between e_1 and e_2 is equal to or less than 1.0 inch.

9.11.3 Raker System

In areas where permission to install tie-back system can not be obtained, or due to other such as Client's preference, the soldier piles can be braced from inside using rakers. The base of the raker system should be embedded a minimum of 2 feet below the lowest adjacent garde. A vertical bearing capacity of 3000 psf can be used in the design of the raker base.

9.11.4 Protection of Reveling and Sloughing

Earth materials exposed on the face of temporary excavation should be kept moist but not saturated to retard reveling and sloughing during construction. In areas with low cohesion soil, lagging will be required for the soil between soldier piles. All lagging should be placed as soon as possible after the excavation is made. If wood lagging is used, care should be taken to fill all void spaces between the excavation face and the lagging. Any materials used for backfill behind the excavation walls should be free-draining. All timber lagging must be removed prior to permanent construction unless the timbers are properly treated. Due to the arching effect of the soils, a lagging pressure of 400 psf may be used for design, providing piles are not spaced larger than 10 feet on centers.

9.11.5 Protection and Monitoring Program

No surcharge loading is allowed within the top five feet of the temporary excavation. Of particular concern is the possibility of heavy construction equipment being placed close to the excavation. Additionally, before placement of equipment close to the excavation edge, CYG should be notified so that any potential change in the lateral soil pressure distribution may be reviewed.

If the proposed temporary excavation should remove lateral support of public street, the designed shoring systems should be approved by the City of Santa Monica and other related agencies. The Contractor should be solely responsible for safety during construction.

It is recommended that a monitoring program for the displacement of the top of temporary excavations and the top of soldier piles be provided the Project Civil Engineer and Shoring Engineer for the observation of possible pile deflection or ground displacement. The monitoring program should include, but not limit to, the following criteria: 1) the method of monitoring, 2) the maximum allowable displacement of the top of temporary excavation, 3) the maximum allowable displacement of the top of soldier piles, and 4) the interval of monitoring.

It is anticipated that major yielding of the adjacent soils may occur during construction. Care should be taken that any movements associated with the yielding are not excessive to harm any underground utilities or adjacent structures. The program of monitoring such movement should be agreed upon the contractors, the site surveyor, the Project Civil Engineer, the project Shoring Engineer and CYG prior to the temporary excavation. It is recommended that the monitoring data be evaluated by CYG to ensure the stability of the temporary excavations. It is also recommended that CYG be allowed to regularly inspect the temporary excavation as work progress in order to monitor earth strain and verify that conditions assumed for design remain unchanged.

All excavations should be stabilized within 30 days of initial excavation. Water should not be allowed to pond on top of the excavations nor to flow toward it.

9.12 Drainage Control

Final grading should provide a positive drainage to divert surface water away from the building foundation and footings in non-erosive devices to the street or other acceptable areas. The building structures should be properly provided with roof gutters and down spouts or equivalent devices. The outlets of down spouts should be either connected to area drains or be extended a minimum of 5 feet away from the building foundation and footings. Underground utility pipes should be absolutely leak free. Landscape watering should be kept to the minimum amount required for vegetation growth.

Subterraneous retaining walls should be provided with a subdrain system at and behind the bottom of each wall. Sump pumps will be required to pump the collected subsurface water to the street or other acceptable areas. All subterraneous retaining walls should be provided with proper waterproofing to prevent the migration of subsurface water or soil moisture to the interior faces of subterraneous walls. Waterproofing system should be designed by the Project Civil Engineer or Structural Engineer.

In order to avoid the migration of a significant amount of surface and subsurface water to foundation soil, the recommendations in this section should be properly incorporated into the design and implemented during construction. The drainage devices should be constantly maintained to ensure proper function.

10.0 PLAN REVIEW AND FIELD INSPECTION/TSTING

The grading plan, shoring plan, foundation plan and retaining wall plan should be reviewed, signed, stamped and approved by CYG prior to the submittal to the City of Los Angeles for final approval.

A City of Los Angeles Grading Deputy Inspector will be required to inspect and approve temporary excavations which closes to property line and removes lateral support from adjacent property.

In accordance with the City of Los Angeles regulations, the following plan review, field inspection and field testing should be performed by CYG to ensure that the recommendations in the geologic and geotechnical engineering reports and the design requirements in the city approved plans are properly implemented.

a. Review, sign and stamp grading plan, shoring plan, foundation plan and retaining wall plan.

b. Inspect and approve temporary excavation and shoring system.

c. Inspect and approve footing excavation and pile drilling.

d. Inspect and approve the bottom to receive fill soil.

e. Inspect and approve the retaining wall subdrain system.

f. Perform fill placement observation and field density test.

A City of Los Angeles Grading Deputy Inspector will be required to inspect and approve temporary excavations which closes to property line and removes lateral support from adjacent property. CYG will provide you City of Los Angeles Grading Deputy Inspector to provide the required inspection for temporary excavations and shoring.

It is the responsibility of you, your representative or your contractor to perform the following issues:

a. Perform all required fill placement and soil compaction.

b. Notify CYG to perform the required inspections and field density tests.

c. Notify the City Deputy Grading Inspector to perform the required inspection.

d. Notify the City Grading Inspector to perform the required inspection.

It is recommended that CYG be notified at least 24 hours prior to any required plan review, site inspection and field testing. All approved plans and permits must be at the job site and available.

A site meeting prior to the commencement of grading is recommended to coordinate the grading activities and required inspections and testings.

11.0 LIMITS AND LIABILITY

The conclusions and recommendations submitted in this report are based on the findings of our data research, subsurface exploration, laboratory testing, engineering evaluation, and engineering analysis. The nature and extent of variations in subsurface conditions may not become evident until construction. If variations then appear evident, it will be necessary to reevaluate the recommendations of this report.

This report is issued with the understanding that it is the responsibility of the Owner, or of his representative to ensure that the recommendations of this report are properly incorporated into the design plan and that the necessary steps are taken to see that the contractors carry out such recommendations in the field.

This report was prepared by CYG for the exclusive use of the client and authorized agent and should not be considered transferable.

This report has been prepared in accordance with generally accepted geotechnical engineering practices. No other warranties either expressed or implied are made as to the professional advice provided under the terms of the agreement for this investigation.

12.0 NOTE

Please be aware that the contract fee for our service to prepare this report does not include any post-report consultation such as addendum report, plan review, field inspection and field testing. When any additional service is required and requested, you will be billed on an hourly basis as shown on the Agreement Form.

APPENDIX A
FIELD EXPLORATION AND LABORATORY TESTING

1.0 FIELD EXPLORATION

Field exploration was performed by one of our engineers on June 11 and 12, 2007 with the aid of a hollow-stem drill rig. Eight (8) deep borings were drilled to a maximum depth of 85 feet at the locations as shown on Plate 1 for liquefaction and foundation evaluations. The borings were logged by the engineer and backfilled on the same day of drilling. The boring logs are presented in Plates A-1 through A-8.

The earth materials encountered in the borings were sampled by using a split-tube soil sampler and a SPT soil sampler. The SPT soil samples were collected by using a 140-pound hammer to drive the SPT standard tube 18 inches into the soil. The falling head for SPT hammer was 30 inches. The blow count values were taken for every 6-inch penetration. The total blow count for the last 12 inches of penetrating distance was recorded as SPT N value. The SPT samples of onsite earth materials were logged and then retained in plastic bags for laboratory particle size tests.

The ring samples of onsite earth materials were logged and then retained in a series of brass rings, each having an inner diameter of 2.4 inches and a height of 1 inch. The ring samples and brass rings were retained in plastic, close-fitting, moisture-tight containers. A bulk sample of onsite soil was collected for laboratory compaction test and expansion index test.

2.0 LABORATORY TEST

Laboratory testing was performed after review of field data and in consideration of the proposed development and the probable foundation and footings to be utilized. The testing procedures of ASTM Standards were followed in testing. The following engineering properties of onsite earth materials were determined: 1) field density and field moisture content, 2) maximum dry density and optimum moisture content, 3) cohesion and friction angle, 4) compressibility and hydroconsolidation, 5) expansion index test, and 6) particle size test.

2.1 Moisture-Density Test

Onsite earth materials were classified in the field and laboratory in accordance with the USCS (Unified Soil Classification System) classification system. Moisture contents are performed in general accordance with ASTM Test Designation D2216-98. Unit weights were determined in general accordance with ASTM Test Designation D2937-04. The results of moisture-density tests are listed in Table A.1.

2.2 Direct Shear Test

Nineteen (19) direct shear tests were performed on selected ring samples to determine the shear strength parameters of alluvial soil. The direct shear tests were performed in accordance with ASTM Standard D-3080-04 by using a strain control type direct shear machine and under an artificially saturated condition. The samples were submerged into water for one or two days to saturate the soil samples prior to testing. The samples were tested under the following procedures: 1) the soil sample is placed in the shear box and then a selected normal stress is applied to the specimen, 2) the soil sample is compressed by the normal stress until an equilibrium state is reached, 3) the soil sample is sheared under a constant rate of shear displacement of 0.004 inches per minute, 4) the peak value of shear strength during shearing was recorded as the peak shear strength, 5) back-shear the soil sample to the original position and then reshear the soil sample to record the peak value as the ultimate shear strength. Three soil samples were tested with different normal loads following the abovementioned testing procedures. The results were plotted on a normal-stress vs. shearing strength diagram to determine the shear strength parameters: cohesion and angle of internal friction. The results of direct shear tests are presented on Plates DS-1 through DS-21.

2.3 Consolidation Test

Twenty seven (27) consolidation tests were performed on selected ring samples to determine the compressibility and hydroconsolidation potential of alluvial soil. The consolidation tests were performed in general accordance with ASTM Standard D-2435-04. The ring samples were soil samples contained in a 2.4-inch-diameter and 1.0-inch-high sampling ring. These tests were performed primarily on materials which would be most susceptible to consolidation under anticipated foundation loading. The samples were tested under the following procedures: 1) the soil sample is placed in a loading frame under a seating pressure of 200 psf, 2) apply vertical loads to the sample in several geometric increments and record the resulting deformations at selected time intervals, 3) adds water to the test cell and records the vertical consolidation when the applied stress reaches a simulated foundation pressure (often 2000 psf) and the sample has consolidated under that pressure, 4) repeat step 2 until a loading pressure of 4000 psf or 8000 psf and record the equilibrium consolidation, 5) unload the sample to an applied stress of 1000 psf and record the rebound of the sample. The results of these tests are presented in terms of percent volume change versus applied vertical stress. The results of consolidation tests are presented on Plates CS-1 through CS-6.

2.4 Compaction Test

Three (3) compaction tests were performed on three bulk soil samples to determine the maximum dry density and optimum moisture content of alluvial soil. The compaction tests were performed in general accordance with ASTM Test Designation D1557-02. The procedure A of compaction test was used in the subject project. The following materials and criteria were followed in test: 1) soil sample passing No.4 sieve was used in test, 2) a 4-inch mode was used in test, 3) a 10-pound hammer with a free fall distance of 18 inches was used in test, 4) five layers of soil sample were compacted in the 4-inch mode, 5) the blow for each layer of soil sample is 25. A minimum of three soil samples were performed to determine the corresponding dry density and moisture content. The results of the tests are presented in terms of moisture content verses dry density to generate a compaction curve. The maximum dry density and optimum moisture content can be determined from the compaction curve. The results of the compaction tests are presented on Plates CM-1 through CM-3.

2.5 Expansion Index Test

Three (3) expansion index tests were performed three bulk soil samples to determine the expansion potential of alluvial soil. The expansion index tests were performed in general accordance with expansion test procedures in ASTM D4829-03 to provide an assessment of the potential for expansion or heave that could be detrimental to foundation or slab performance. The following procedures were followed in the test: 1) compact the soil sample at degree of saturation between 45 and 55 percent in a 4.01-inch-diameter, 1.0-inch-high ring, 2) apply a vertical seating pressure of 144 psf to the sample, 3) add water to the test cell and saturate the soil sample, 4) record the soil expansion until the expansion of soil sample stops. The volume of swell is converted to an expansion index. The expansion index test indicated that expansion indexes of 3, 4 and 26 for the tested alluvial soil. Soils with an expansion index in the range of 0 to 20 are considered as very low expansive soils. Soils with an expansion index in the range of 21 to 50 are considered as low expansive soils.

2.6 Sieve Analysis and Hydrometer Test

Ten (10) mechanic sieve tests and six (6) hydrometer tests were performed on selected soil samples in accordance with ASTM Standard D-422-63 (1998) to determine the grain size distributions of onsite soil. Mechanic sieve analyses establish gradation for the coarse-grained particles (i.e. sand and gravel). Hydrometer tests establish gradation for the fine-grained particles (i.e. silt and clay). The results of gradation analyses are presented on Plates GD-1 through GD-10.

Table A.1. Results of Dry Density-Moisture Content Tests

Location	Depth ft	Soil Description	Dry Density pcf	Moisture Content, %	SPT (N) Blow Count
B-1	5	Grayish brown sand (Qa)	98	22	27
B-1	10	Grayish brown silty sand (Qa)	106	3	28
B-1	15	Grayish brown silty sand (Qa)	107	4	23
B-1	20	Grayish brown silty sand (Qa)	101	7	25
B-1	25	Brown silty sand (Qa)	114	5	27
B-1	30	Brown clayey sandy silt (Qa)	119	12	8
B-1	35	Brown clayey sandy silt (Qa)	118	16	10
B-1	40	Brown gravelly sand (Qa)	136	8	61
B-1	45	Grayish brown gravelly sand (Qa)	125	5	53
B-1	50	Grayish brown gravelly sand (Qa)	120	5	44
B-1	55	Brown silty sand (Qa)	120	15	33
B-1	60	Dark brown silty sand (Qa)	120	16	35
B-1	65	Brown silty sand (Qa)	118	16	38
B-1	70	Grayish brown gravelly sand (Qa)	123	5	59
B-1	75	Grayish brown gravelly sand (Qa)	117	5	90
B-2	1	Grayish brown silty sand (Qa)	120	7	----
B-2	6	Dark grayish brown silty sand (Qa)	104	9	----
B-2	11	Grayish brown silty sand (Qa)	100	7	----
B-2	16	Grayish brown sandy silt (Qa)	98	14	----
B-2	21	Grayish brown silty sand (Qa)	99	5	----
B-2	26	Brown clayey sandy silt (Qa)	114	12	----
B-2	31	Brown sandy silt (Qa)	115	12	----
B-2	36	Light grayish brown gravelly sand (Qa)	112	4	----
B-2	41	Grayish brown gravelly sand (Qa)	118	3	----
B-2	46	Grayish brown gravelly sand (Qa)	113	4	----
B-2	51	Grayish brown gravelly sand (Qa)	114	2	----
B-2	56	Brown clayey silt (Qa)	116	18	----
B-2	61	Light grayish brown gravelly sand (Qa)	126	6	----
B-2	66	Brown sandy silty clay (Qa)	119	15	----
B-2	71	Grayish brown silty sand (Qa)	107	4	----

B-2	75	Light grayish brown gravelly sand (Qa)	109	2	----
B-3	2	Grayish brown silty sand (Qa)	122	10	----
B-3	7	Grayish brown gravelly silty sand (Qa)	103	2	----
B-3	12	Grayish brown silty sand (Qa)	93	14	----
B-3	17	Grayish brown silty sand (Qa)	110	5	----
B-3	22	Grayish brown silty sand (Qa)	109	2	----
B-3	27	Brown sandy silt (Qa)	110	10	----
B-3	32	Grayish brown gravelly silty sand (Qa)	121	6	----
B-3	37	Grayish brown gravelly silty sand (Qa)	110	4	----
B-3	42	Grayish brown gravelly silty sand (Qa)	108	6	----
B-3	47	Grayish brown gravelly silty sand (Qa)	109	4	----
B-3	52	Grayish brown gravelly silty sand (Qa)	110	3	----
B-3	57	Dark brown sandy clayey silt (Qa)	119	14	----
B-3	62	Light grayish brown gravelly sand (Qa)	109	6	----
B-3	67	Brown sandy silt (Qa)	120	14	----
B-4	3	Light brown clayey silty sand (Qa)	119	2	----
B-4	13	Light brown gravelly sand (Qa)	113	2	----
B-4	18	Brown silty sand (Qa)	117	5	----
B-4	23	Brown silty sand (Qa)	109	11	----
B-4	28	Brown clayey sandy silt (Qa)	104	15	----
B-4	33	Brown clayey sandy silt (Qa)	91	11	----
B-4	38	Light brown gravelly sand (Qa)	97	5	----
B-4	43	Light brown gravelly sand (Qa)	129	3	----
B-4	48	Light brown silty sand (Qa)	107	10	----
B-4	53	Light brown clayey silty sand (Qa)	122	12	----
B-4	58	Light brown silty sand (Qa)	112	16	----
B-4	63	Light brown gravelly sand (Qa)	112	3	----
B-4	68	Light brown clayey silty sand (Qa)	122	10	----
B-4	73	Light brown gravelly sand (Qa)	112	6	----
B-4	78	Light brown gravelly sand (Qa)	114	2	----
B-4	83	Light brown gravelly sand (Qa)	119	1	----
B-5	4	Light brown sand (Qa)	108	5	----

B-5	9	Light brown silty sand (Qa)	93	11	----
B-5	32	Light brown sand (Qa)	114	4	----
B-5	34	Light brown sand (Qa)	111	9	----
B-5	39	Light brown clayey silty sand (Qa)	108	13	----
B-5	44	Light brown clayey sandy silt (Qa)	115	19	----
B-5	49	Light brown gravelly sand (Qa)	107	4	----
B-5	54	Light brown sand (Qa)	112	2	----
B-5	59	Brown clayey silty sand (Qa)	120	11	----
B-5	64	Brown clayey sandy silt (Qa)	117	17	----
B-5	69	Brown clayey sandy silt (Qa)	107	23	----
B-5	74	Light brown gravelly sand (Qa)	117	5	----
B-5	79	Light brown gravelly sand (Qa)	103	5	----
B-6	5	Light brown gravelly sand (Qa)	107	6	42
B-6	10	Light brown silty sand (Qa)	104	2	27
B-6	15	Light brown gravelly sand (Qa)	110	2	23
B-6	20	Light brown gravelly sand (Qa)	115	2	27
B-6	25	Light brown gravelly sand (Qa)	126	3	33
B-6	30	Light brown gravelly sand (Qa)	----	----	32
B-6	35	Light brown gravelly sand (Qa)	117	4	35
B-6	40	Light brown sand (Qa)	111	6	55
B-6	45	Light brown sand (Qa)	102	13	74
B-6	50	Light brown sand (Qa)	129	2	74
B-6	55	Brown clayey silty sand (Qa)	122	2	36
B-6	60	Brown silty sand (Qa)	125	8	37
B-6	65	Brown clayey silty sand (Qa)	127	8	40
B-6	70	Brown clayey silty sand (Qa)	124	13	38
B-6	75	Brown clayey silty sand (Qa)	115	12	46
B-6	80	Light brown sand (Qa)	118	3	56
B-6	85	Light brown sand (Qa)	111	4	----
B-7	4	Light brown gravelly sand (Qa)	115	3	----
B-7	9	Light brown sand (Qa)	111	4	----
B-7	14	Light brown clayey sandy silt (Qa)	96	15	----

B-7	19	Light brown silty sand (Qa)	115	7	----
B-7	24	Light brown clayey sandy silt (Qa)	99	19	----
B-7	29	Light brown clayey sandy silt (Qa)	107	17	----
B-7	34	Light brown clayey sandy silt (Qa)	112	16	----
B-7	39	Light brown clayey sandy silt (Qa)	111	4	----
B-7	44	Light brown gravelly sand (Qa)	114	3	----
B-7	49	Light brown silty sand (Qa)	103	18	----
B-7	54	Light brown gravelly sand (Qa)	116	4	----
B-7	59	Light brown silty sand (Qa)	105	18	----
B-7	64	Light brown sand (Qa)	114	6	----
B-7	69	Light brown sand (Qa)	110	5	----
B-7	74	Light brown sand (Qa)	111	4	----
B-7	79	Light brown gravelly sand (Qa)	114	4	----
B-8	5	Light brown gravelly sand (Qa)	114	3	26
B-8	10	Light brown gravelly sand (Qa)	118	2	30
B-8	15	Light brown sand (Qa)	109	3	34
B-8	20	Brown clayey silty sand (Qa)	120	4	36
B-8	25	Brown clayey silty sand (Qa)	119	7	37
B-8	30	Brown clayey sandy silt (Qa)	121	12	10
B-8	35	Brown clayey sandy silt (Qa)	113	16	11
B-8	40	Light brown gravelly sand (Qa)	131	2	43
B-8	45	Light brown gravelly sand (Qa)	125	2	68
B-8	50	Light brown gravelly sand (Qa)	128	3	37
B-8	55	Light brown gravelly sand (Qa)	121	13	35
B-8	60	Light brown silty sand (Qa)	124	6	35
B-8	65	Light brown clayey silty sand (Qa)	128	10	39
B-8	70	Light brown sand (Qa)	118	4	51
B-8	75	Light brown sand (Qa)	117	4	73
B-8	80	Light brown sand (Qa)	112	4	52

BORING LOG

Location	Depth ft	Blow Count (N)	Soils Descriptions
B - 1	0" - 8"		**4" Asphalt with 4" SubBase**
			Alluvium (8" - 76.5')
	8" - 5'		Grayish brown silty sand, slightly moist, moderately dense.
	5' - 10'	27	Grayish brown silty sand, moist, moderately dense.
	10' - 15'	28	Grayish brown silty sand, moist, moderately dense.
	15' - 20'	23	Grayish brown silty sand, slightly moist to moist, moderately dense.
	20' - 25'	25	Grayish brown silty sand, slightly moist, moderately dense.
	25' - 30'	27	Brown silty sand, slightly moist, moderately.
	30' - 35'	8	Brown clayey sandy silt, slightly moist, moderately dense.
	35' - 40'	10	Brown clayey sandy silt, slightly moist to moist, moderately firm.
	40' - 45'	61	Brown gravelly sand, slightly moist to moist, moderately firm to firm.
	45' - 50'	53	Grayish brown gravelly sand, slightly moist, dense.
	50' - 55'	44	Grayish brown gravelly sand, slightly moist, very dense.
	55' - 60'	33	Brown silty sand, slightly moist, firm.
	60' - 65'	35	Dark brown silty sand, slightly moist, stiff.
	65' - 70'	38	Brown silty sand, slightly moist, stiff.
	70' - 75'	59	Grayish brown gravelly sand, slightly moist, dense to very dense.
	75' - 76.5'	90	Grayish brown gravelly sand, slightly moist, dense to very dense.
			Ends at 76.5 ft. No water. No caving. Samples at 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60, 65, 70, 75 ft.

Plate A-1

BORING LOG

Location	Depth ft	Soils Descriptions
B - 2	0" - 10"	**6" Asphalt with 4" SubBase**
		Alluvium (10" - 76')
	10" - 6'	Grayish brown silty sand, slightly moist, moderately dense.
	6' - 11'	Dark grayish brown silty sand, slightly moist, moderately dense.
	11' - 16'	Grayish brown silty sand, slightly moist to moist, moderately dense.
	16' - 21'	Grayish brown sandy silt, slightly moist to moist, moderately firm.
	21' - 26'	Grayish brown silty sand, slightly moist to moist, moderately dense.
	26' - 31'	Brown clayey sandy silt, slightly moist to moist, moderately firm.
	31' - 36'	Brown sandy silt, slightly moist to moist, moderately firm to firm.
	36' - 41'	Light grayish brown gravelly sand, slightly moist to moist, dense.
	41' - 46'	Grayish brown gravelly sand, slightly moist, dense to very dense.
	46' - 51'	Grayish brown gravelly sand, slightly moist, very dense.
	51' - 56'	Grayish brown gravelly sand, slightly moist, very dense.
	56' - 61'	Brown clayey silt, moist, moderately firm to firm.
	61' - 66'	Light grayish brown gravelly sand, slightly moist, very dense.
	66' - 71'	Brown sandy silty clay, slightly moist to moist, stiff to very stiff.
	71' - 75'	Grayish brown silty sand, slightly moist to moist, dense.
	75' - 76'	Light grayish brown gravelly sand, slightly moist, very dense.
		Ends at 76 ft. No water. No caving. Samples at 1, 6, 11, 16, 21, 26, 31, 36, 41, 46, 51, 56, 61, 66, 71, 75 ft.

Plate A-2

BORING LOG

Location	Depth ft	Soils Descriptions
B - 3	0" - 8"	**4" Asphalt with 4" SubBase**
		Alluvium (8" - 73')
	8" - 2'	Grayish brown silty sand, slightly moist, moderately dense.
	2' - 7'	Grayish brown silty sand, slightly moist, moderately dense.
	7' - 12'	Grayish brown gravelly silty sand, slightly moist, moderately dense.
	12' - 17'	Grayish brown silty sand, slightly moist, moderately dense.
	17' - 22'	Grayish brown silty sand, slightly moist to moist, moderately dense.
	22' - 27'	Grayish brown silty sand, slightly moist, moderately dense.
	27' - 32'	Brown sandy silt, slightly moist, moderately firm to firm.
	32' - 37'	Grayish brown gravelly silty sand, slightly moist to moist, moderately dense.
	37' - 42'	Grayish brown gravelly silty sand, slightly moist, dense.
	42' - 47'	Grayish brown gravelly silty sand, slightly moist, dense.
	47' - 52'	Grayish brown gravelly silty sand, slightly moist, dense.
	52' - 57'	Grayish brown gravelly silty sand, slightly moist, dense.
	57'- 62'	Dark brown sandy clayey silt, slightly moist to moist,
	62' - 67'	Light grayish brown gravelly sand, slightly moist, dense.
	67' - 72	Brown sandy silt, moist, moderately firm to firm.
	72' - 73'	Light brown gravelly sand, slightly moist, dense.
		Ends at 73 ft. No water. No caving. Samples at 2, 7, 12, 17, 22, 27, 32, 37, 42, 47, 52, 57, 62, 67 ft.

Plate A-3

BORING LOG

Location	Depth ft	Soils Descriptions
B - 4	0" - 4"	**Asphalt**
		Alluvium (4" - 85')
	4" - 8'	Light brown clayey silty sand, slightly moist, moderately dense.
	8' - 18'	Light brown gravelly sand, slightly moist, moderately dense.
	18' - 28'	Brown silty sand, slightly moist, moderately dense.
	28' - 33'	Brown clayey sandy silt, slightly porous, moist, moderately firm.
	33' - 38'	Brown clayey sandy silty to clayey silty sand, slightly porous, moist, moderately dense.
	38' - 48'	Light brown gravelly sand, moist, dense.
	48' - 53'	Light brown silty fine sand, moist, dense.
	53' - 58'	Light brown clayey silty sand, moist, moderately dense.
	58' - 63'	Light brown silty sand, moist, moderately dense.
	63' - 68'	Light brown gravelly sand, slightly moist, moderately dense to dense.
	68' - 73'	Light brown clayey silty sand, moist, dense.
	73' - 85'	Light brown sand with occasional gravels, moist, dense.
		Ends at 85 ft. No water. No caving. Samples at 3, 13, 18, 23, 28, 33, 38, 43, 48, 53, 58, 63, 68, 73, 78, 83 ft.

Plate A-4

BORING LOG

Location	Depth ft	Soils Descriptions
B - 5	0" - 4"	**Asphalt**
		Alluvium (4" - 80')
	4" - 4'	Light brown gravelly silty sand, slightly moist, moderately dense.
	4' - 9'	Light brown sand, slightly moist, moderately dense.
	9' - 14'	Light brown silty sand, moist, moderately dense.
	14' - 39'	Light brown sand, contains occasional gravels, moist, moderately dense.
	39' - 44'	Light brown clayey silty sand, moist, moderately dense.
	44' - 49'	Light brown clayey sandy silt, moist, moderately dense.
	49' - 54'	Light brown gravelly sand, moist, dense.
	54' - 59'	Light brown sand, moist, dense.
	59' - 64'	Brown clayey silty sand, moist, dense.
	64' - 74'	Brown clayey sandy silt, moist, firm.
	74' - 80'	Light brown gravelly sand, moist, dense.
		Ends at 80 ft. No water. No caving Samples at 4, 9, 32, 34, 39, 44, 49, 54, 59, 64, 69, 74, 79 ft.

Plate A-5

BORING LOG

Location	Depth ft	Blow Count (N)	Soils Descriptions
B - 6	0" - 3"		**Asphalt**
	3" - 1'		**Artificial fill (3" - 1')** Light brown gravelly silty sand, slightly moist, moderately dense.
	1' - 5'		**Alluvium (1' - 85')** Light brown silty sand, moist, moderately dense.
	5' - 10'	42	Light brown gravelly sand, moist, dense.
	10' - 15'	27	Light brown silty sand, moist, moderately dense.
	15' - 20'	23	Light brown gravelly sand, slightly moist, moderately dense
	20' - 25'	27	Light brown gravelly sand, slightly moist, moderately dense
	25' - 30'	23	Light brown gravelly sand, slightly moist, moderately dense
	30' - 35'	32	Light brown gravelly sand, slightly moist, moderately dense
	35' - 40'	35	Light brown gravelly sand, slightly moist, moderately dense
	40' - 45'	35	Light brown sand, slightly moist, moderately dense.
	45' - 50'	74	Light brown sand, slightly moist, moderately dense.
	50' - 55'	74	Light brown sand, slightly moist, moderately dense.
	55' - 60'	36	Brown clayey silty sand, moist, moderately dense.
	60' - 65'	37	Brown silty sand, moist, moderately dense.
	65' - 70'	40	Brown clayey silty sand, moist, moderately dense.
	70' - 75'	38	Brown clayey silty sand, moist, moderately dense.
	75' - 80'	46	Brown clayey silty sand, moist, moderately dense.
	80' - 85'	56	Light brown sand, moist, dense.
			Ends at 85 ft. No water. No caving. Samples at 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60, 65, 70, 75, 80, 85 ft.

Plate A-6

BORING LOG

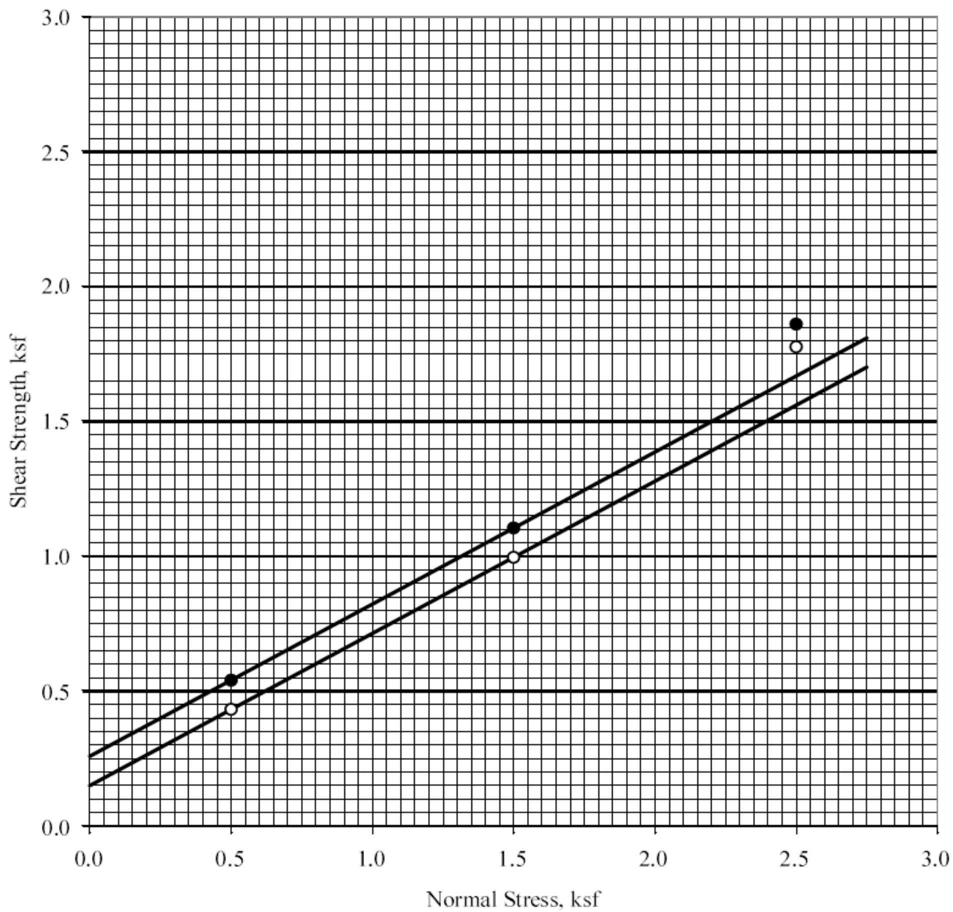
Location	Depth ft	Soils Descriptions
B - 7	0" - 4"	**Asphalt**
		Alluvium (4" - 80')
	4" - 9'	Light brown gravelly sand, moist, moderately dense.
	9' - 14'	Light brown sand, moist, moderately dense.
	14' - 19'	Light brown clayey sandy silt, moist, moderately firm.
	19' - 24'	Light brown silty sand, moist, moderately dense.
	24' - 44'	Light brown clayey sandy silt, porous, moist, moderately firm.
	44' - 49'	Light brown gravelly sand, moist, dense.
	49' - 54'	Light brown silty sand, moist, moderately dense.
	54' - 59'	Light brown gravelly sand, moist, dense.
	59' - 64'	Light brown silty fine sand, moist, moderately dense.
	64' - 79'	Light brown sand, moist, dense.
	79' - 80'	Light brown gravelly sand, moist, dense.
		Ends at 80 ft. No water. No caving. Samples at 4, 9, 14, 19, 24, 29, 34, 39, 44, 49, 54, 59, 64, 69, 74, 79 ft.

Plate A-7

BORING LOG

Location	Depth ft	Blow Count (N)	Soils Descriptions
B - 8	0" - 3"		**Asphalt**
	3" - 5'		**Alluvium (3" - 82')** Light brown gravelly silty sand, moist, moderately dense.
	5' - 10'	26	Light brown gravelly sand, moist, moderately dense.
	10' - 15'	30	Light brown gravelly sand, moist, moderately dense.
	15' - 20'	34	Light brown sand, moist, moderately dense
	20' - 25'	36	Brown clayey silty sand, moist, moderately dense.
	25' - 30'	37	Brown clayey silty sand, moist, moderately dense.
	30' - 35'	10	Brown clayey sandy silt, moist, moderately firm.
	35' - 40'	11	Brown clayey sandy silt, moist, moderately firm.
	40' - 45'	43	Light brown gravelly sand, moist, dense.
	45' - 50'	68	Light brown gravelly sand, moist, dense.
	50' - 55'	37	Light brown gravelly sand, moist, dense.
	55' - 60'	35	Light brown gravelly sand to sand, moist, moderately dense.
	60' - 65'	35	Light brown silty sand moist, moderately dense.
	65' - 70'	39	Light brown clayey silty sand, moist, moderately dense.
	70' - 75'	51	Light brown sand, moist, moderately dense.
	75' - 80'	73	Light brown sand, contains occasional gravels, moist, moderately dense.
	80' - 82'	52	Light brown sand, contains occasional gravels, moist, moderately dense.
			Ends at 82 ft. No water. No caving. Samples at 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60, 65, 70, 75, 80 ft.

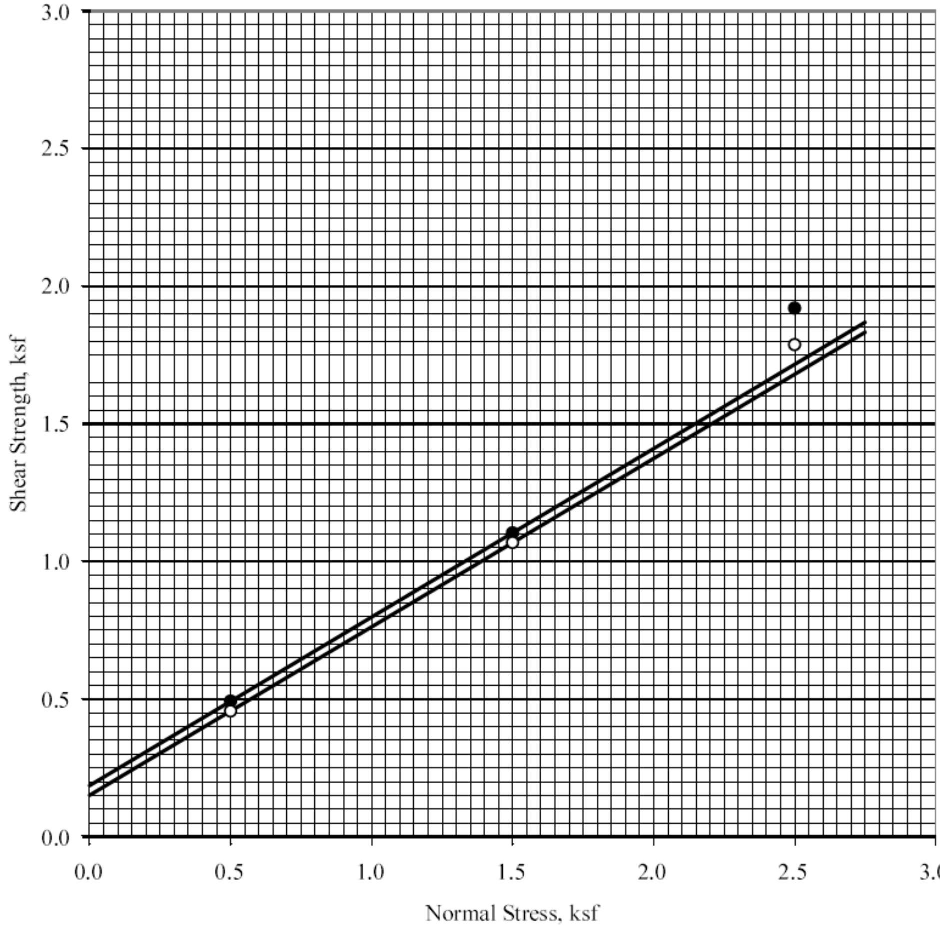
Plate A-8



● Peak	- At Saturation Moisture Content	C = 250 psf	ϕ = 29 °
○ Ultimate	- At Saturation Moisture Content	C = 150 psf	ϕ = 29 °

Field Dry Density = 98 pcf Field Moisture Content = 22 % Saturation Moisture Content = 27 %	Boring : B-1 Depth : 5 feet Description : Grayish brown sand (Qa)
C. Y. GEOTECH, INC. Geotechnical Engineering and Engineering Geology	Gilbert Salazar Date : 06-2007 P.N. No.: CYG-07-4948 **Shear Diagram**

Plate DS - 1



● Peak - At Saturation Moisture Content C = 180 psf ϕ = 31 °

○ Ultimate - At Saturation Moisture Content C = 150 psf ϕ = 31 °

Field Dry Density = 107 pcf
Field Moisture Content = 4 %
Saturation Moisture Content = 21 %

Boring : B-1
Depth : 15 feet
Description : Grayish brown sand (Qa)

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Geotechnical Engineering
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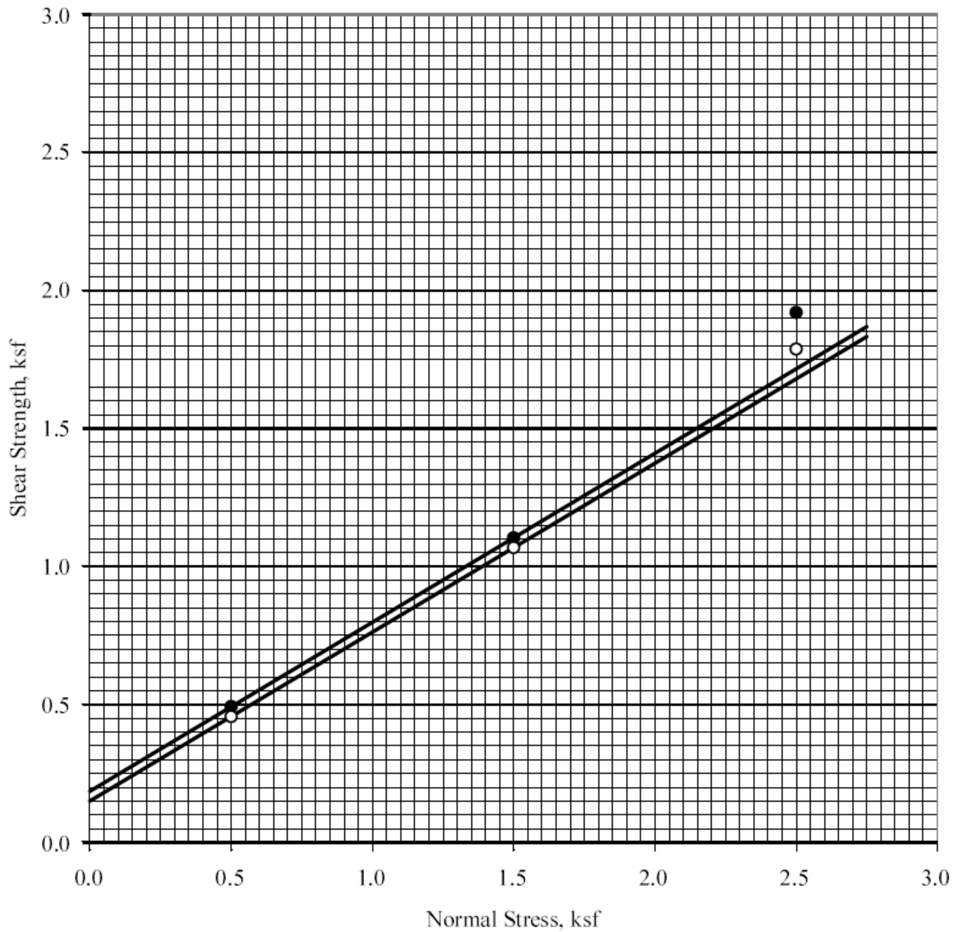
Gilbert Salazar

Date : 06-2007 P.N. No.: CYG-07-4948

Shear Diagram

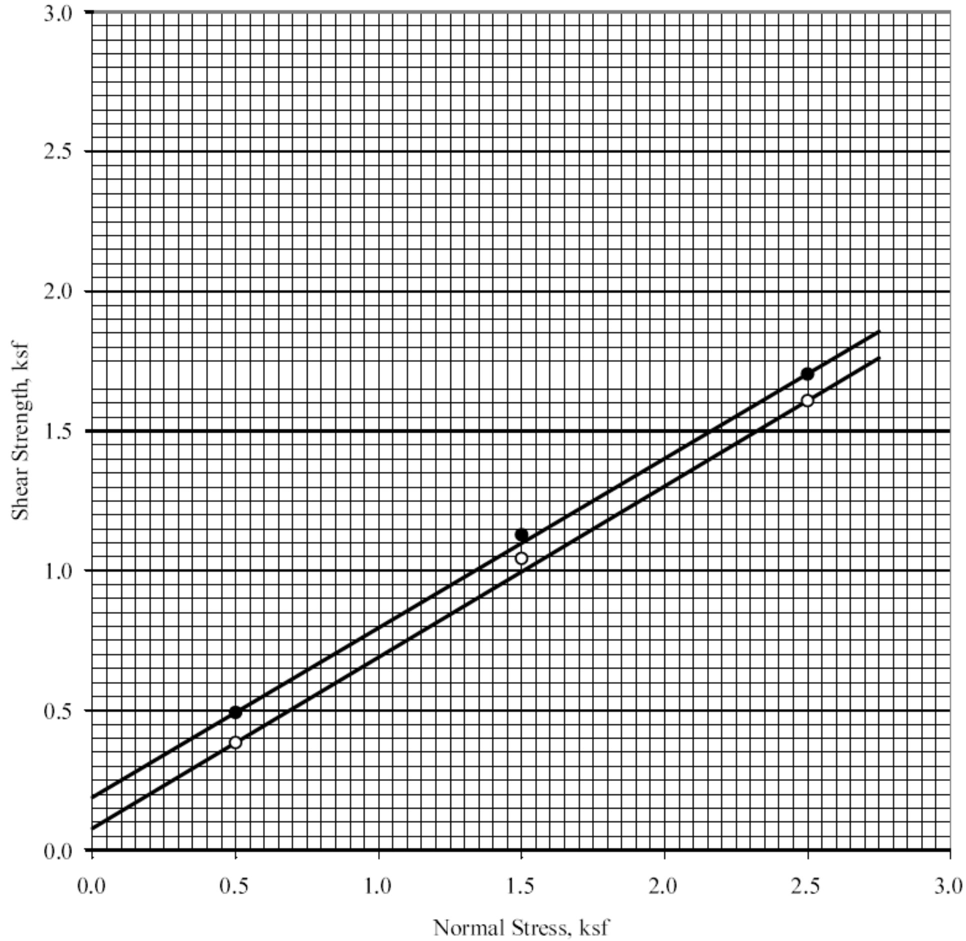
Plate DS - 2



● Peak	- At Saturation Moisture Content	C = 180 psf	ϕ = 31 °
○ Ultimate	- At Saturation Moisture Content	C = 150 psf	ϕ = 31 °

Field Dry Density = 107 pcf Field Moisture Content = 4 % Saturation Moisture Content = 21 %	Boring : B-1 Depth : 15 feet Description : Grayish brown sand (Qa)
C. Y. GEOTECH, INC. Geotechnical Engineering and Engineering Geology	Gilbert Salazar Date : 06-2007 P.N. No.: CYG-07-4948 **Shear Diagram**

Plate DS - 2



● Peak	- At Saturation Moisture Content	C = 180 psf	ϕ = 31 °
○ Ultimate	- At Saturation Moisture Content	C = 70 psf	ϕ = 31 °

Field Dry Density = 119 pcf
Field Moisture Content = 12 %
Saturation Moisture Content = 15 %

Boring : B-1
Depth : 30 feet
Description : Brown clayey sandy silt (Qa)

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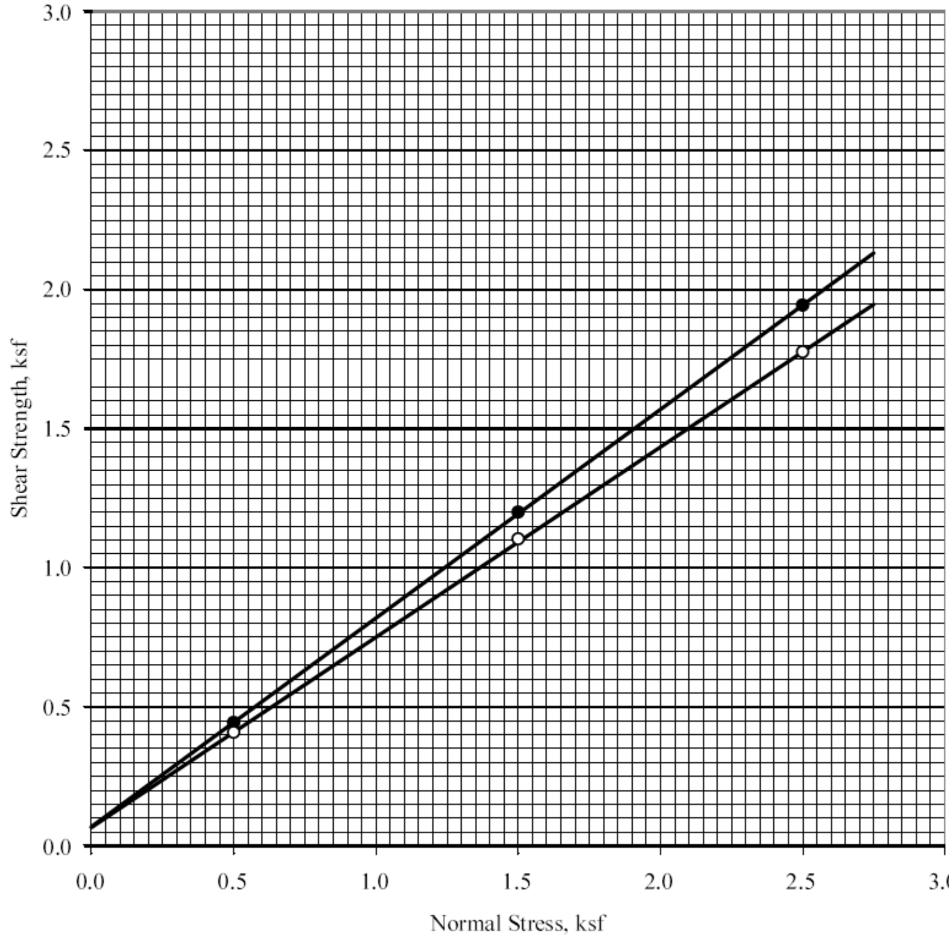
Gilbert Salazar

Date : 06-2007 P.N. No.: CYG-07-4948

Shear Diagram

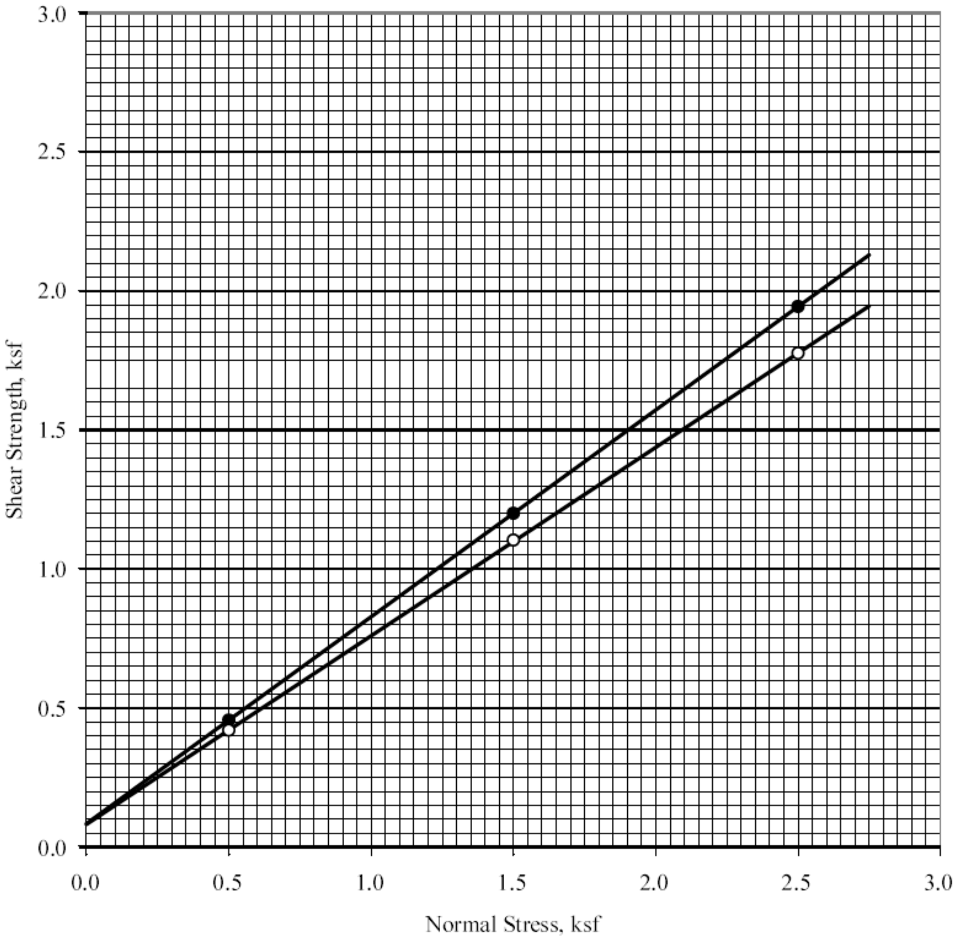
Plate DS - 4



● Peak	- At Saturation Moisture Content	C = 60 psf	ϕ = 36.5 °
○ Ultimate	- At Saturation Moisture Content	C = 60 psf	ϕ = 34 °

Field Dry Density = 104 pcf Field Moisture Content = 9 % Saturation Moisture Content = 23 %	Boring : B-2 Depth : 6 feet Description : Dark grayish brown silty sand (Qa)
C. Y. GEOTECH, INC. Geotechnical Engineering and Engineering Geology	Gilbert Salazar Date : 09-2007 — P.N. No.: CYG-07-4948 **Shear Diagram**

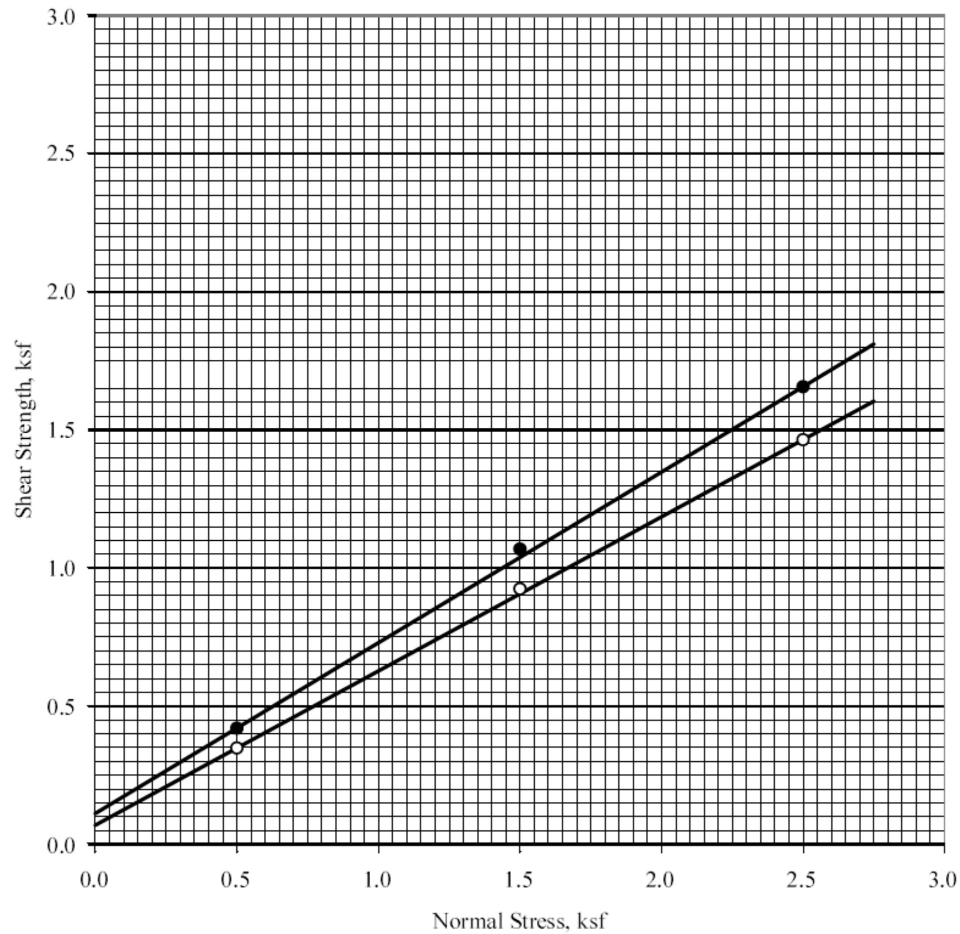
Plate DS - 5



● Peak - At Saturation Moisture Content C = 80 psf ϕ = 36.5 °

○ Ultimate - At Saturation Moisture Content C = 80 psf ϕ = 34 °

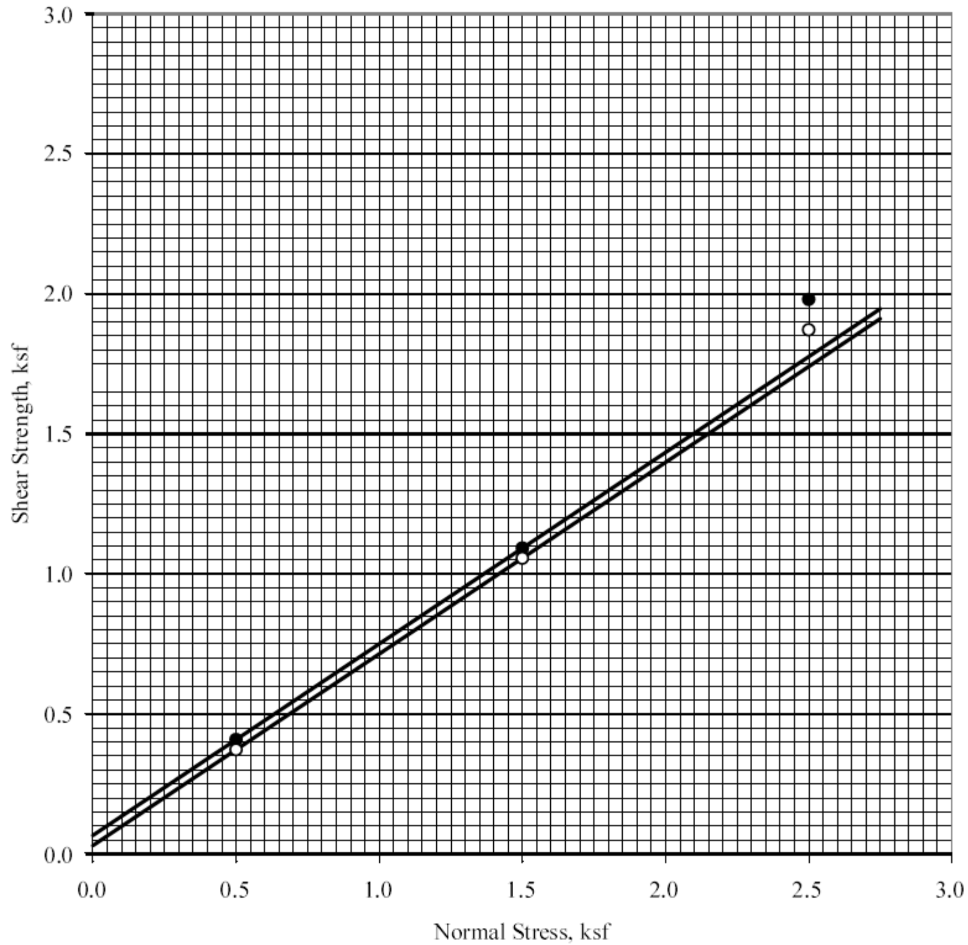
Field Dry Density = 99 pcf Field Moisture Content = 5 % Saturation Moisture Content = 26 %	Boring : B-2 Depth : 21 feet Description : Grayish brown silty sand (Qa)
C. Y. GEOTECH, INC. Geotechnical Engineering and Engineering Geology	Gilbert Salazar Date : 09-2007 / P.N. No.: CYG-07-4948 **Shear Diagram**



● Peak - At Saturation Moisture Content C = 110 psf ϕ = 31.5 °
○ Ultimate - At Saturation Moisture Content C = 60 psf ϕ = 29 °

Field Dry Density = 112 pcf Field Moisture Content = 4 % Saturation Moisture Content = 19 %	Boring : B-2 Depth : 36 feet Description : Light grayish brown gravelly sand (Qa)
C. Y. GEOTECH, INC. Geotechnical Engineering and Engineering Geology	Gilbert Salazar Date : 09-2007 P.N. No.: CYG-07-4948 **Shear Diagram**

Plate DS - 7



● Peak - At Saturation Moisture Content C = 60 psf ϕ = 34 °

○ Ultimate - At Saturation Moisture Content C = 30 psf ϕ = 34 °

Field Dry Density = 97 pcf Field Moisture Content = 5 % Saturation Moisture Content = 27 %	Boring : B-4 Depth : 38 feet Description : Light brown gravelly sand (Qa)

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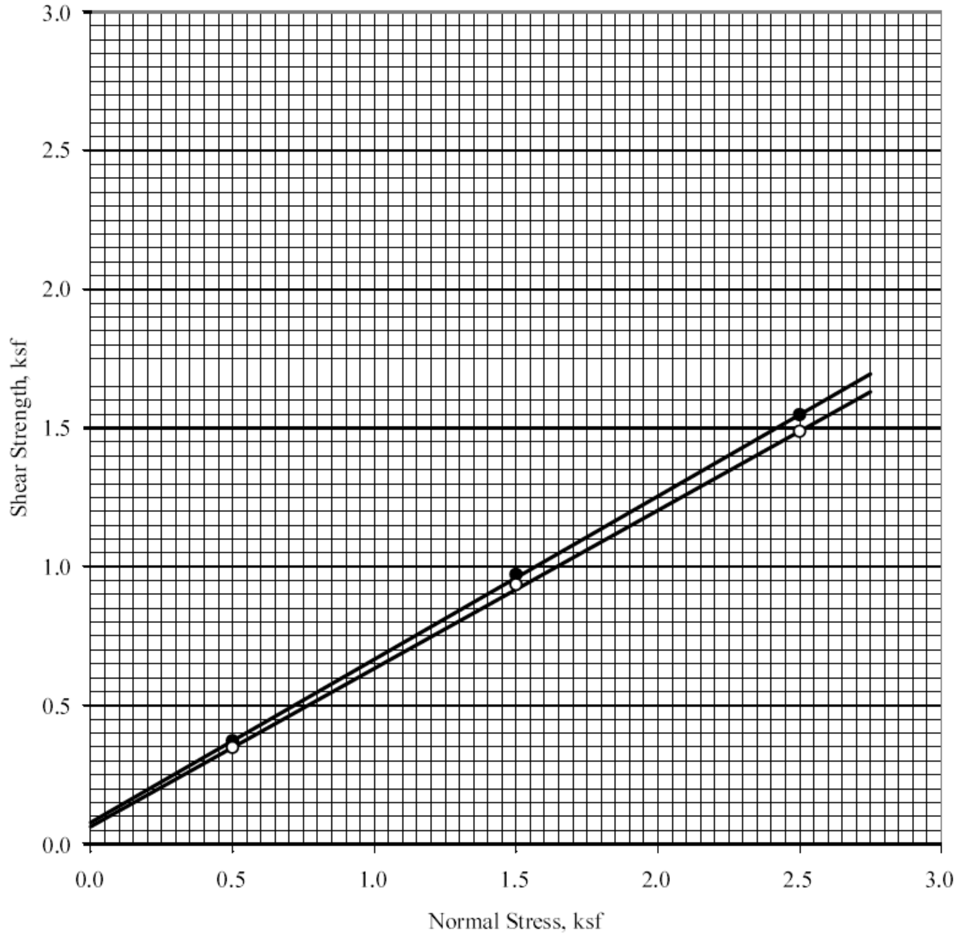
Gilbert Salazar

Date : 09-2007 P.N. No.: CYG-07-4948

Shear Diagram



● Peak	- At Saturation Moisture Content	C = 70 psf	ϕ = 30 °
○ Ultimate	- At Saturation Moisture Content	C = 60 psf	ϕ = 29.5 °

Field Dry Density = 117 pcf
Field Moisture Content = 4 %
Saturation Moisture Content = 16 %

Boring : B-6
Depth : 35 feet
Description : Light brown gravelly sand (Qa)

C. Y. GEOTECH, INC.

Geotechnical Engineering
and Engineering Geology

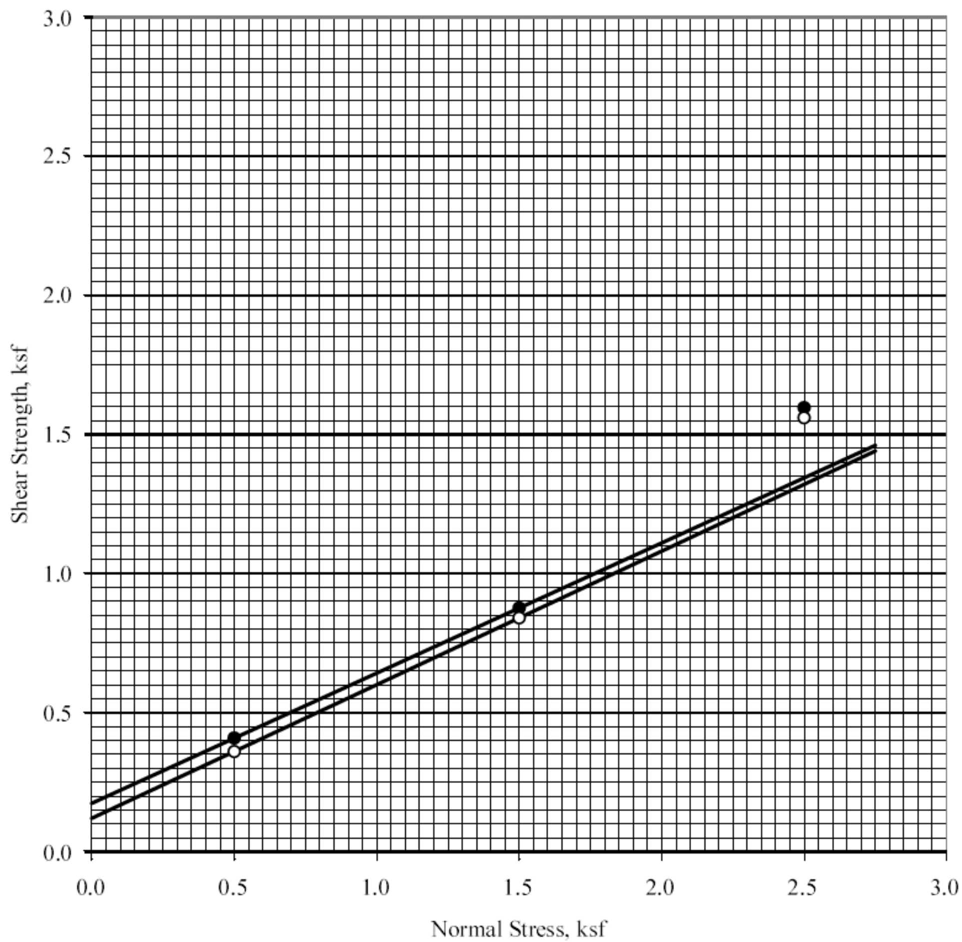
Gilbert Salazar

Date : 09-2007

P.N. No.: CYG-07-4948

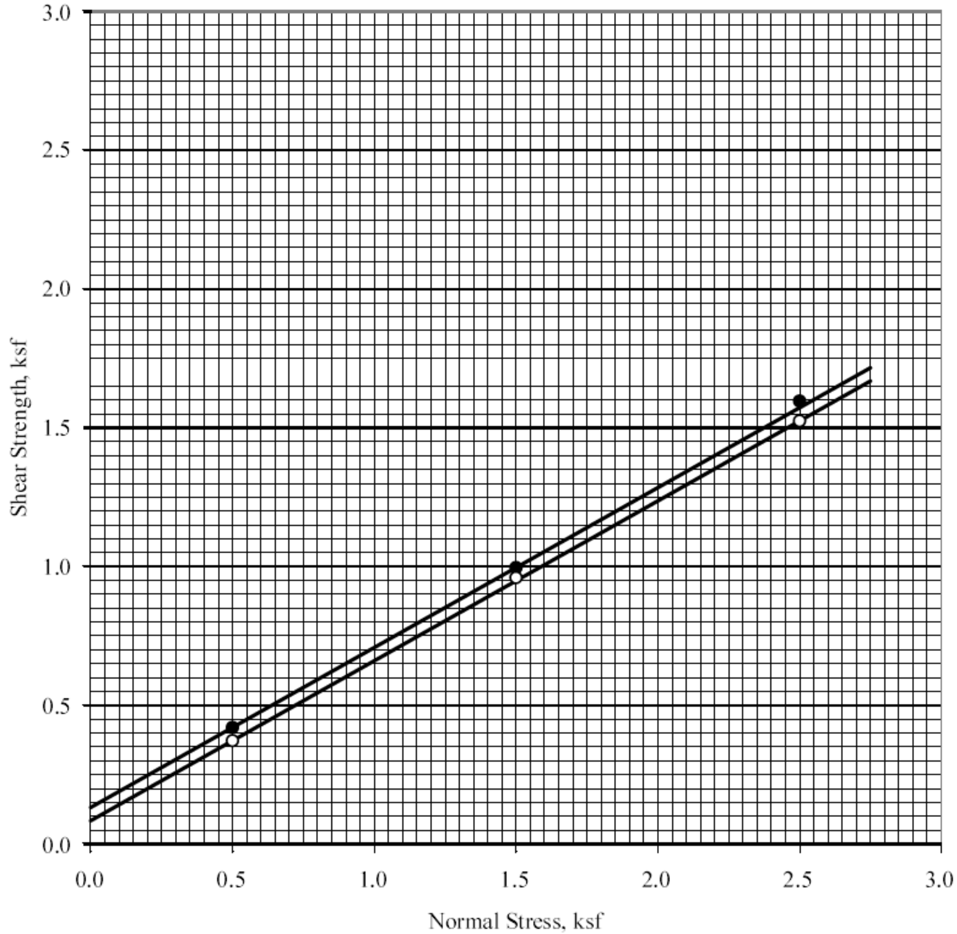
Shear Diagram

Plate DS - 9



● Peak - At Saturation Moisture Content C = 170 psf ϕ = 25 °
○ Ultimate - At Saturation Moisture Content C = 120 psf ϕ = 25.5 °

Field Dry Density = 111 pcf Field Moisture Content = 4 % Saturation Moisture Content = 19 %	Boring : B-7 Depth : 9 feet Description : Light brown sand (Qa)
C. Y. GEOTECH, INC. Geotechnical Engineering and Engineering Geology	Gilbert Salazar Date : 09-2007 — P.N. No.: CYG-07-4948 **Shear Diagram**



● Peak	- At Saturation Moisture Content	C = 130 psf	ϕ = 29.5 °
○ Ultimate	- At Saturation Moisture Content	C = 80 psf	ϕ = 29.5 °

Field Dry Density = 99 pcf
Field Moisture Content = 19 %
Saturation Moisture Content = 26 %

Boring : B-7
Depth : 24 feet
Description : Light brown clayey sandy silt (Qa)

C. Y. GEOTECH, INC.

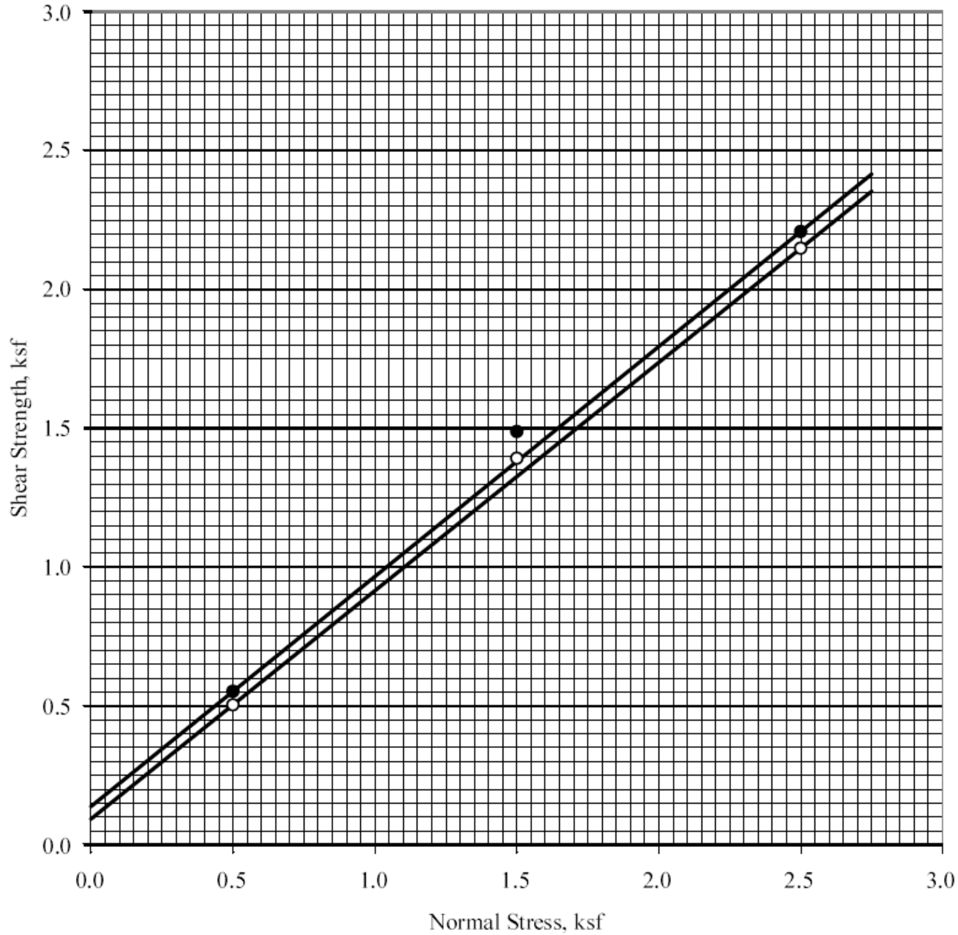
Geotechnical Engineering
and Engineering Geology

Gilbert Salazar

Date : 09-2007

P.N. No.: CYG-07-4948

Shear Diagram

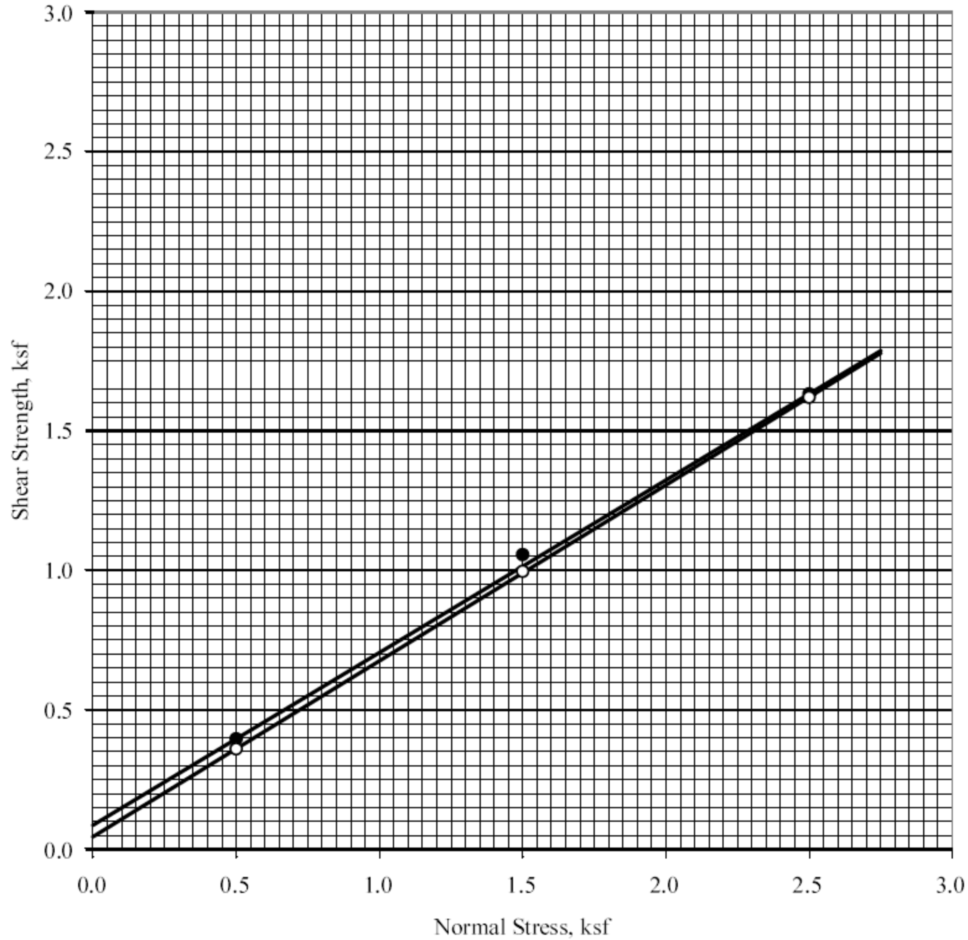


● Peak - At Saturation Moisture Content C = 130 psf ϕ = 39.5 °

○ Ultimate - At Saturation Moisture Content C = 90 psf ϕ = 39 °

Field Dry Density = 114 pcf Field Moisture Content = 3 % Saturation Moisture Content = 18 %	Boring : B-7 Depth : 44 feet Description : Light brown gravelly sand (Qa)
C. Y. GEOTECH, INC. Geotechnical Engineering and Engineering Geology	Gilbert Salazar Date : 09-2007 — P.N. No.: CYG-07-4948 **Shear Diagram**

Plate DS - 12



● Peak - At Saturation Moisture Content C = 80 psf ϕ = 31.5 °

○ Ultimate - At Saturation Moisture Content C = 40 psf ϕ = 32 °

Field Dry Density = 120 pcf
Field Moisture Content = 4 %
Saturation Moisture Content = 15 %

Boring : B-8
Depth : 20 feet
Description : Brown clayey silty sand (Qa)

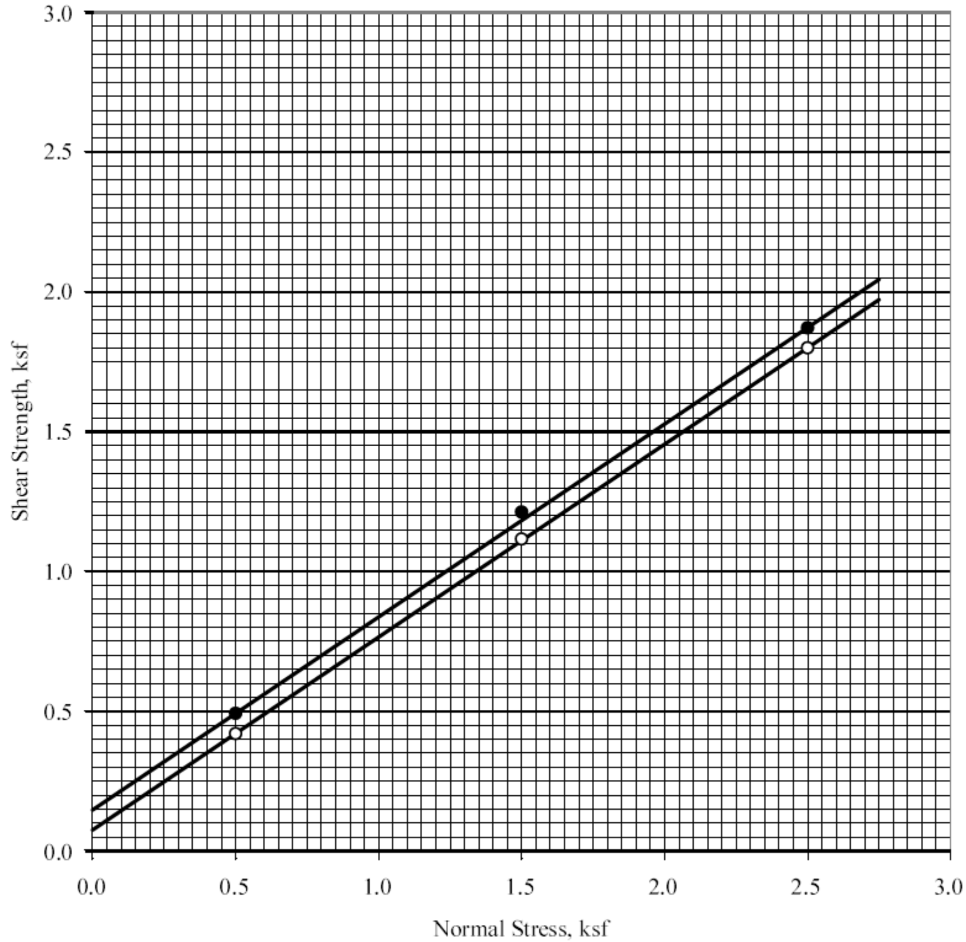
C. Y. GEOTECH, INC.

Geotechnical Engineering
and Engineering Geology

Gilbert Salazar

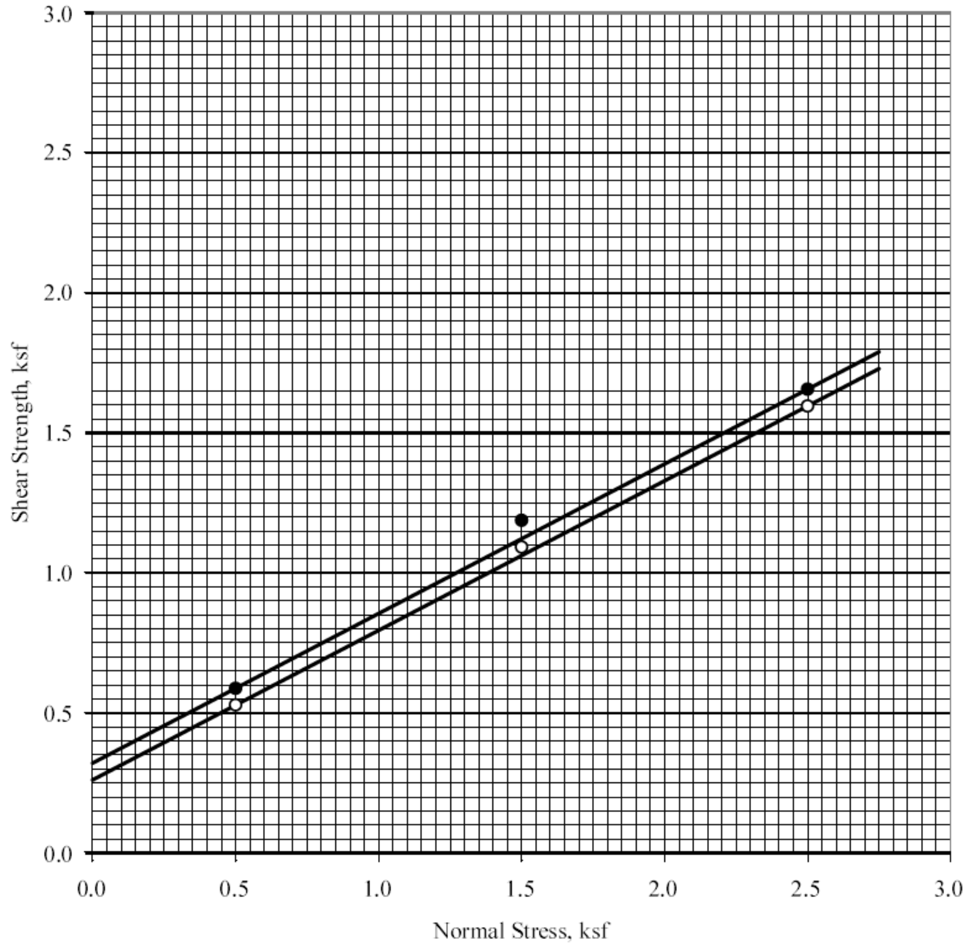
Date : 09-2007 P.N. No.: CYG-07-4948

Shear Diagram



● Peak	- At Saturation Moisture Content	C = 140 psf	ϕ = 34.5 °
○ Ultimate	- At Saturation Moisture Content	C = 70 psf	ϕ = 34.5 °

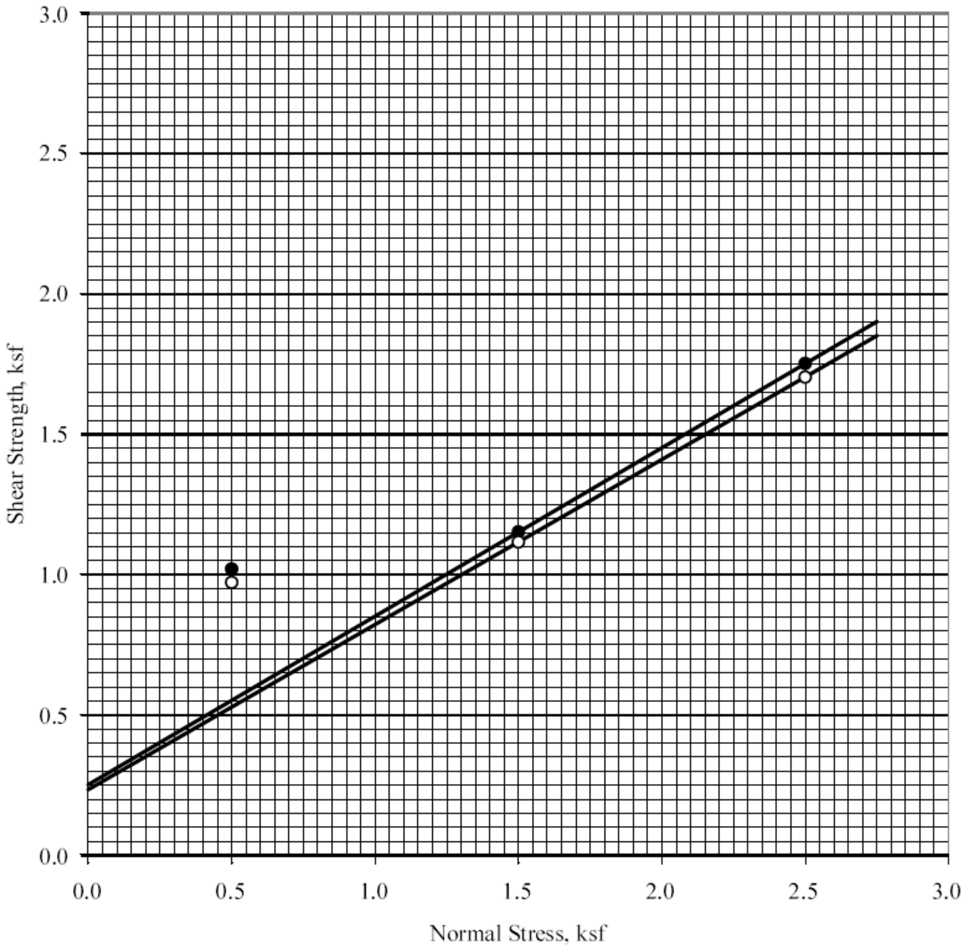
Field Dry Density = 113 pcf Field Moisture Content = 16 % Saturation Moisture Content = 18 %	Boring : B-8 Depth : 35 feet Description : Brown clayey sandy silt (Qa)
C. Y. GEOTECH, INC. Geotechnical Engineering and Engineering Geology	Gilbert Salazar Date : 09-2007 — P.N. No.: CYG-07-4948 **Shear Diagram**



● Peak	- At Saturation Moisture Content	C = 320 psf	ϕ = 28 °
○ Ultimate	- At Saturation Moisture Content	C = 260 psf	ϕ = 28 °

Field Dry Density = 125 pcf Field Moisture Content = 5 % Saturation Moisture Content = 13 %	Boring : B-1 Depth : 45 feet Description : Grayish brown gravelly sand (Qa)
C. Y. GEOTECH, INC. Geotechnical Engineering and Engineering Geology	Gilbert Salazar Date : 09-2007 / P.N. No.: CYG-07-4948 **Shear Diagram**

Plate DS - 15

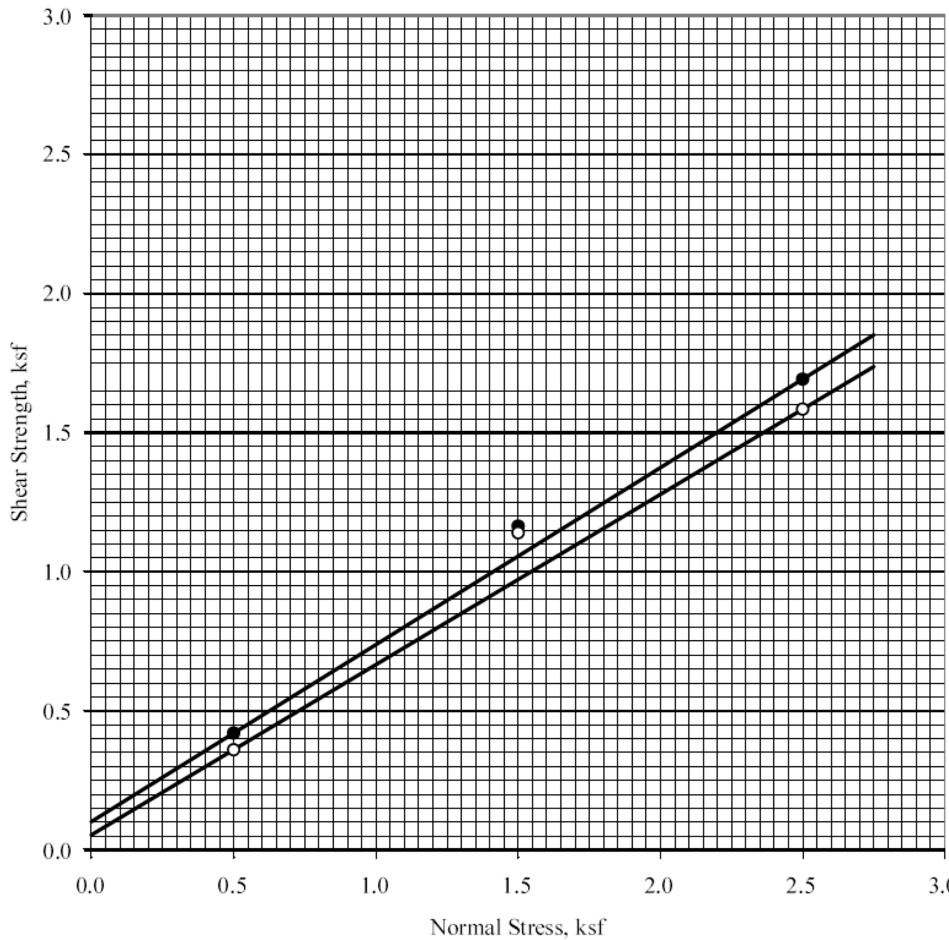


● Peak - At Saturation Moisture Content C = 250 psf ϕ = 30.5 °

○ Ultimate - At Saturation Moisture Content C = 230 psf ϕ = 30 °

Field Dry Density = 114 pcf Field Moisture Content = 2 % Saturation Moisture Content = 17 %	Boring : B-2 Depth : 51 feet Description : Grayish brown gravelly sand (Qa)
C. Y. GEOTECH, INC. Geotechnical Engineering and Engineering Geology	Gilbert Salazar Date : 09-2007 P.N. No.: CYG-07-4948 **Shear Diagram**

Plate DS - 16



● Peak	- At Saturation Moisture Content	C = 100 psf	ϕ = 32 °
○ Ultimate	- At Saturation Moisture Content	C = 50 psf	ϕ = 31 °

Field Dry Density = 108 pcf
Field Moisture Content = 6 %
Saturation Moisture Content = 20 %

Boring : B-3
Depth : 42 feet
Description : Grayish brown gravelly silty sand (Qa)

C. Y. GEOTECH, INC.

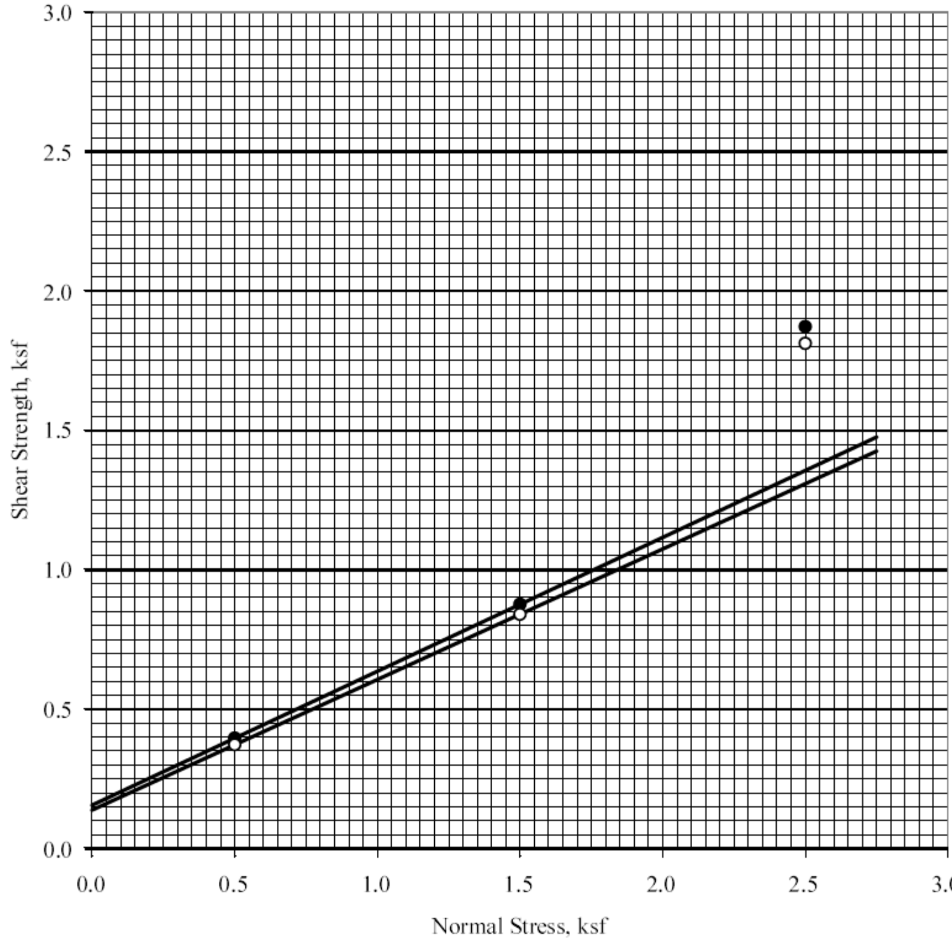
Geotechnical Engineering
and Engineering Geology

Gilbert Salazar

Date : 09-2007 P.N. No.: CYG-07-4948

Shear Diagram

Plate DS - 17



● Peak	- At Saturation Moisture Content	C = 150 psf	ϕ = 25.5 °
○ Ultimate	- At Saturation Moisture Content	C = 130 psf	ϕ = 25 °

Field Dry Density = 107 pcf
Field Moisture Content = 4 %
Saturation Moisture Content = 21 %

Boring : B-5
Depth : 49 feet
Description : Light brown gravelly sand (Qa)

C. Y. GEOTECH, INC.

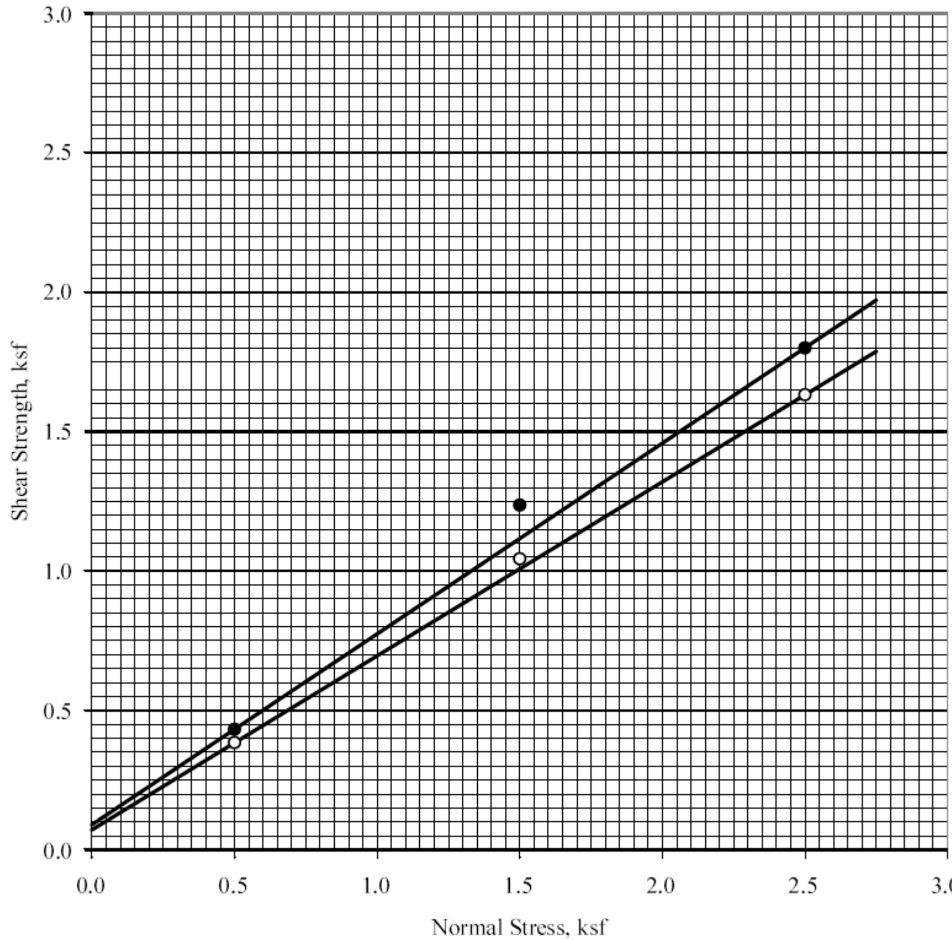
Geotechnical Engineering and Engineering Geology

Gilbert Salazar

Date : 09-2007 | P.N. No.: CYG-07-4948

Shear Diagram

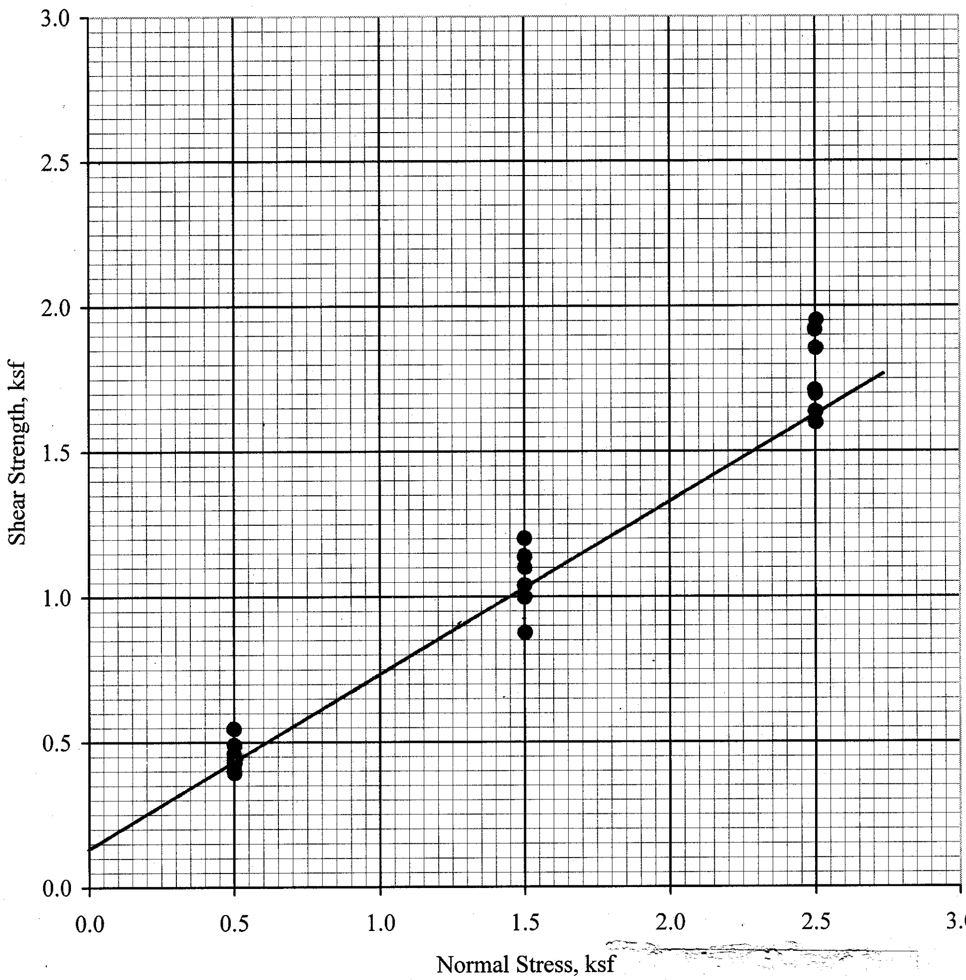
Plate DS - 18



● Peak - At Saturation Moisture Content C = 80 psf ϕ = 34 °

○ Ultimate - At Saturation Moisture Content C = 70 psf ϕ = 31.5 °

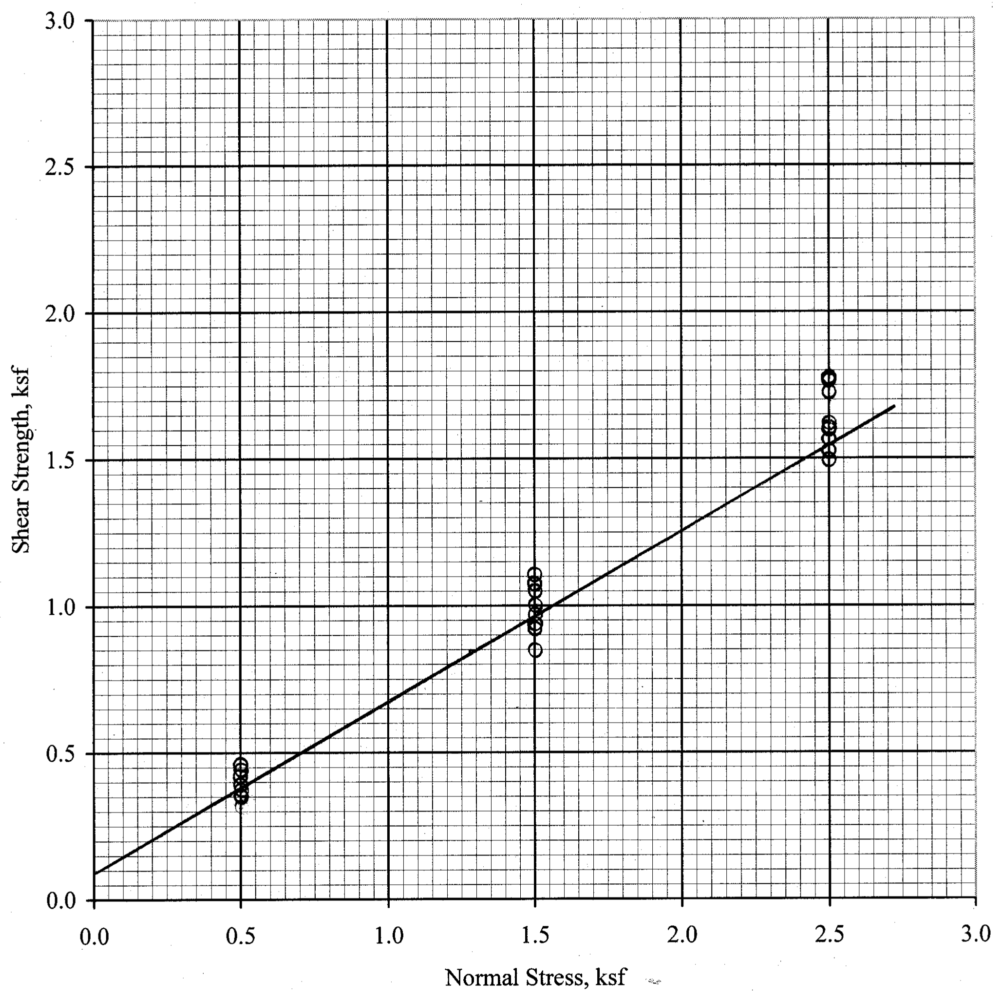
Field Dry Density = 122 pcf Field Moisture Content = 2 % Saturation Moisture Content = 14 %	Boring : B-6 Depth : 55 feet Description : Brown clayey silty sand (Qa)
C. Y. GEOTECH, INC. Geotechnical Engineering and Engineering Geology	Gilbert Salazar Date : 09-2007 P.N. No.: CYG-07-4948 **Shear Diagram**



● Peak - At Saturation Moisture Content C = 140 psf ϕ = 31 °

○ Ultimate - At Saturation Moisture Content C = psf ϕ = °

Field Dry Density = pcf Field Moisture Content = % Saturation Moisture Content = %	Boring : Onsite Depth : 0 -30 feet Description : Summary of Peak Shear Strength (0 -30 feet)
C. Y. GEOTECH, INC. Geotechnical Engineering and Engineering Geology	Galbert Salazar Date : 09-2007	P.N. No.: CYG-07-4948 **Shear Diagram**



● Peak - At Saturation Moisture Content — C = 140 psf, ϕ = [illegible] °

○ Ultimate - At Saturation Moisture Content — C = 90 psf, ϕ = 30.5 °

Field Dry Density = pcf Field Moisture Content = % Saturation Moisture Content = %	Boring : Onsite Depth : 0 -30 feet Description : Summary of Ultimate Shear Strength (0 -30 feet)
C. Y. GEOTECH, INC. Geotechnical Engineering and Engineering Geology	Galbert Salazar
	Date : 09-2007 — P.N. No.: CYG-07-4948
	Shear Diagram

C. Y. GEOTECH, INC. Geotechnical Engineering and Engineering Geology		Consolidation Test
Gilbert Salazar		
Date : 06-2007	P.N. No: CYG-07-4948	

Boring	Depth (feet)	Water Content (%) Before	After	Height (inches)	Diameter (inches)
B-1	30	12	15	1.0	2.4

Classification : Brown clayey sandy silt (Qa)
Hydroconsolidation = 0.1 %

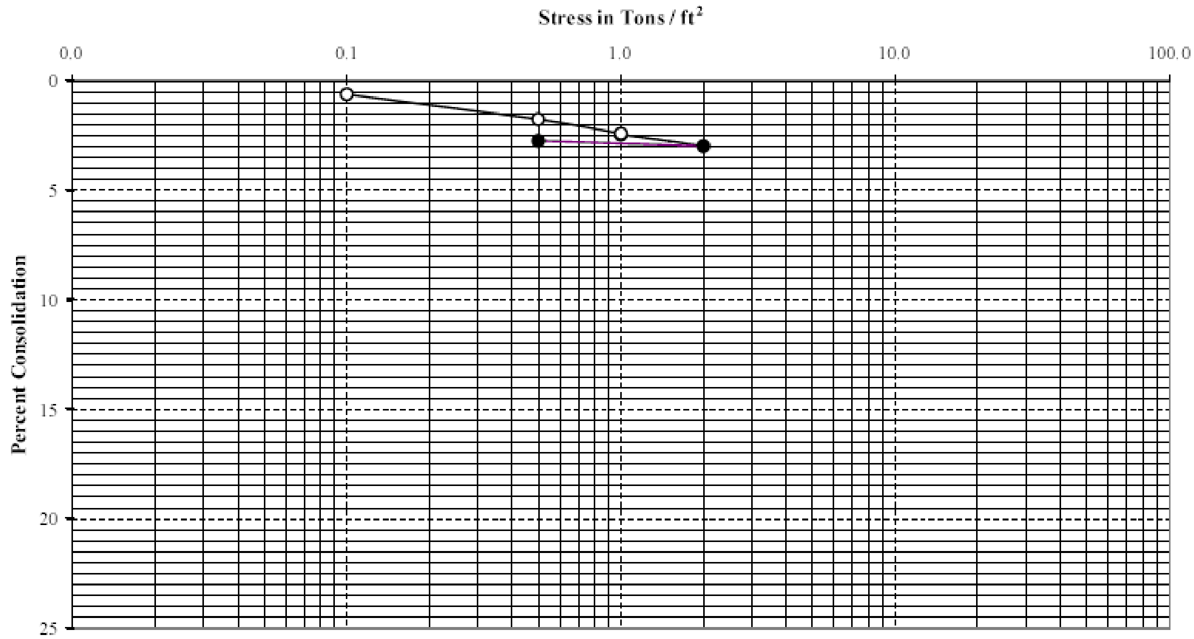


Plate CS - 1

C. Y. GEOTECH, INC.

Geotechnical Engineering and Engineering Geology

Gilbert Salazar

Date : 06-2007 | P.N. No: CYG-07-4948

Consolidation Test

Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
B-1	40	8	9	1.0	2.4

Classification : Brown gravelly sand (Qa)

Hydroconsolidation = 0.5 %

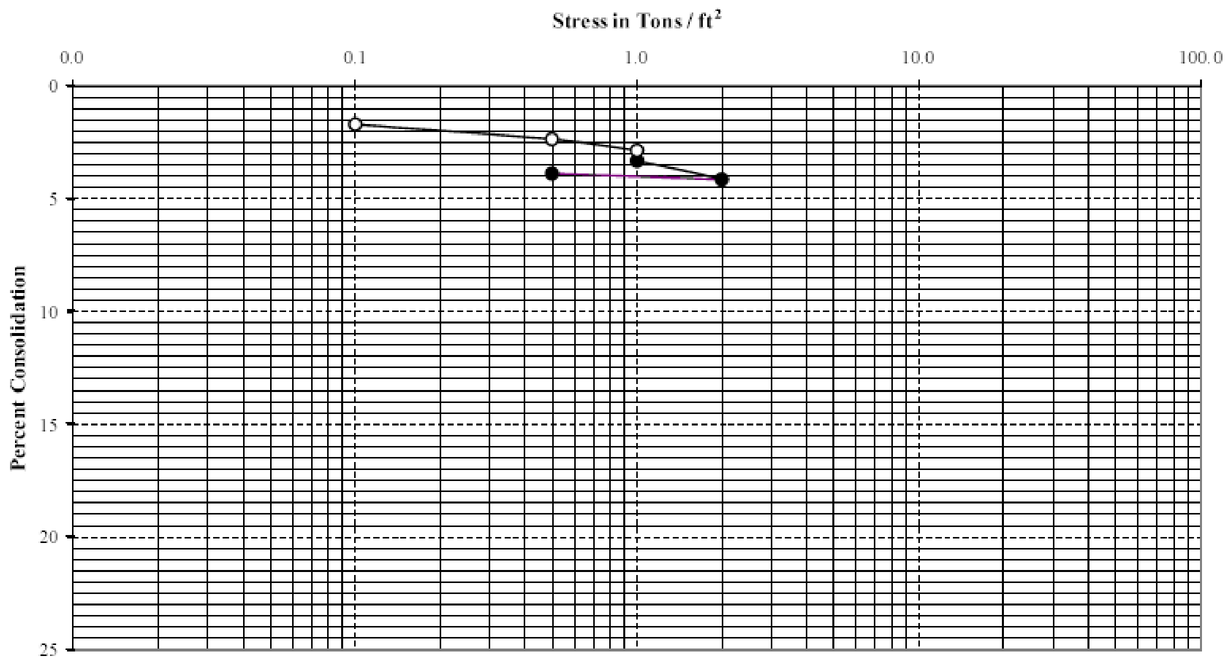


Plate CS - 2

C. Y. GEOTECH, INC.

Geotechnical Engineering and Engineering Geology

Gilbert Salazar

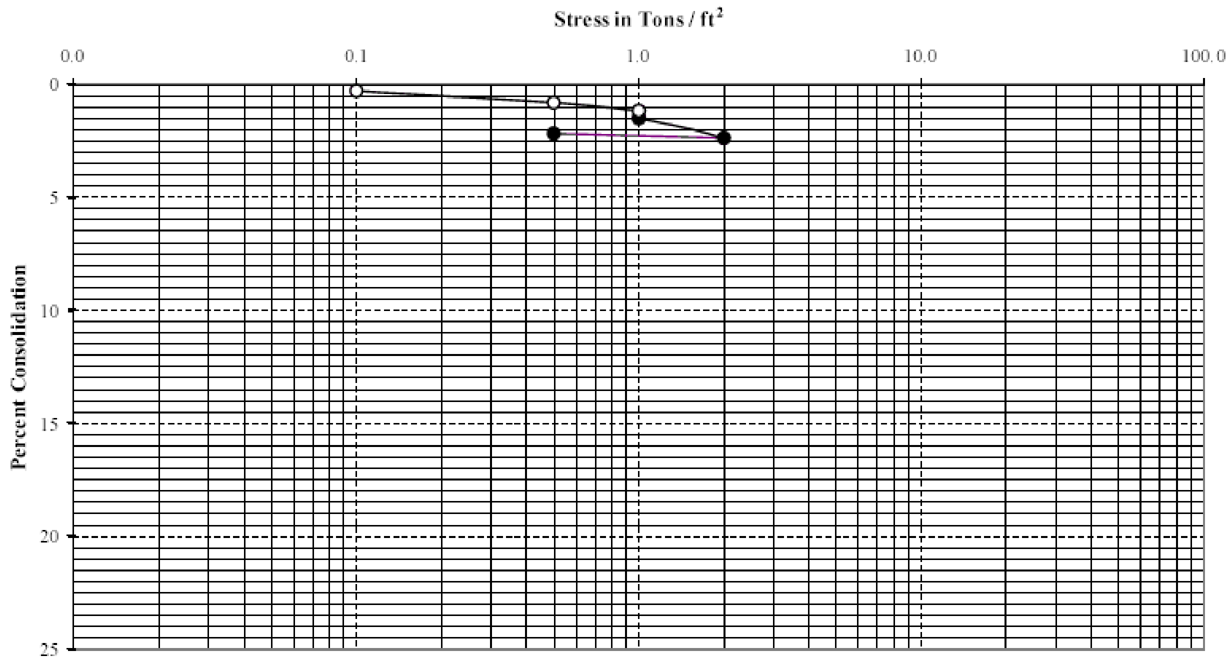
Date : 09-2007 | P.N. No: CYG-07-4948

Consolidation Test

Boring	Depth (feet)	Water Content (%) Before	After	Height (inches)	Diameter (inches)
B-1	50	5	15	1.0	2.4

Classification : Grayish brown gravelly sand (Qa)

Hydroconsolidation = 0.3 %



C. Y. GEOTECH, INC.
Geotechnical Engineering and Engineering Geology

Gilbert Salazar

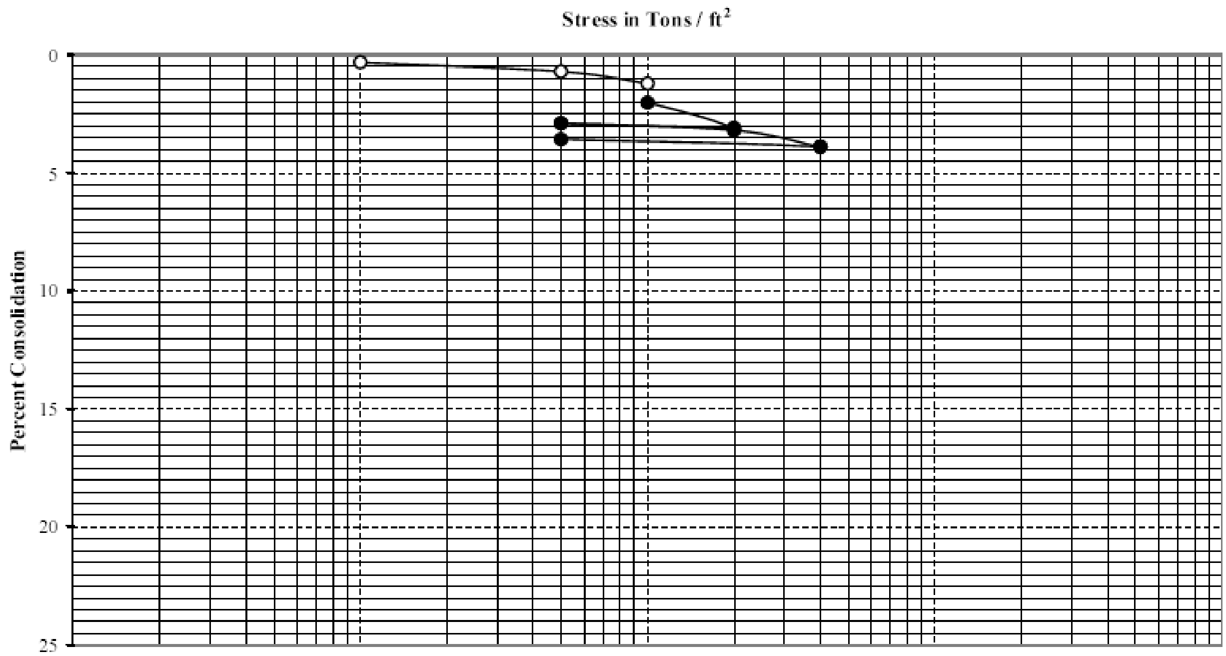
Date : 09-2007 | P.N. No: CYG-07-4948

Consolidation Test

Boring	Depth (feet)	Water Content (%) Before	After	Height (inches)	Diameter (inches)
B-1	70	5	14	1.0	2.4

Classification : Grayish brown gravelly sand (Qa)

Hydroconsolidation = 0.8 %



C. Y. GEOTECH, INC.
Geotechnical Engineering and Engineering Geology

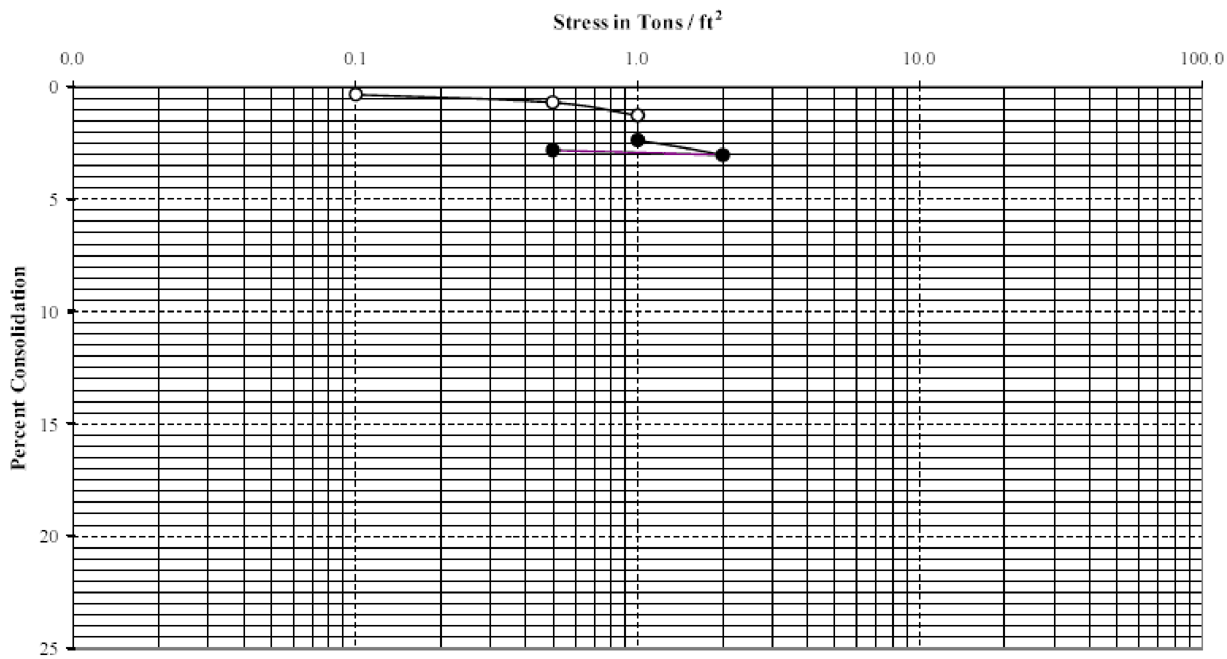
Gilbert Salazar

Date : 09-2007 | P.N. No: CYG-07-4948

Consolidation Test

Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
B-2	31	12	17	1.0	2.4

Classification : Brown sandy silt (Qa)
Hydroconsolidation = 1.1 %



C. Y. GEOTECH, INC.

Geotechnical Engineering and Engineering Geology

Gilbert Salazar

Date : 09-2007 | P.N. No: CYG-07-4948

Consolidation Test

Boring	Depth (feet)	Water Content (%) Before	After	Height (inches)	Diameter (inches)
B-2	41	3	16	1.0	2.4

Classification : Grayish brown gravelly sand (Qa)

Hydroconsolidation = 0.2 %

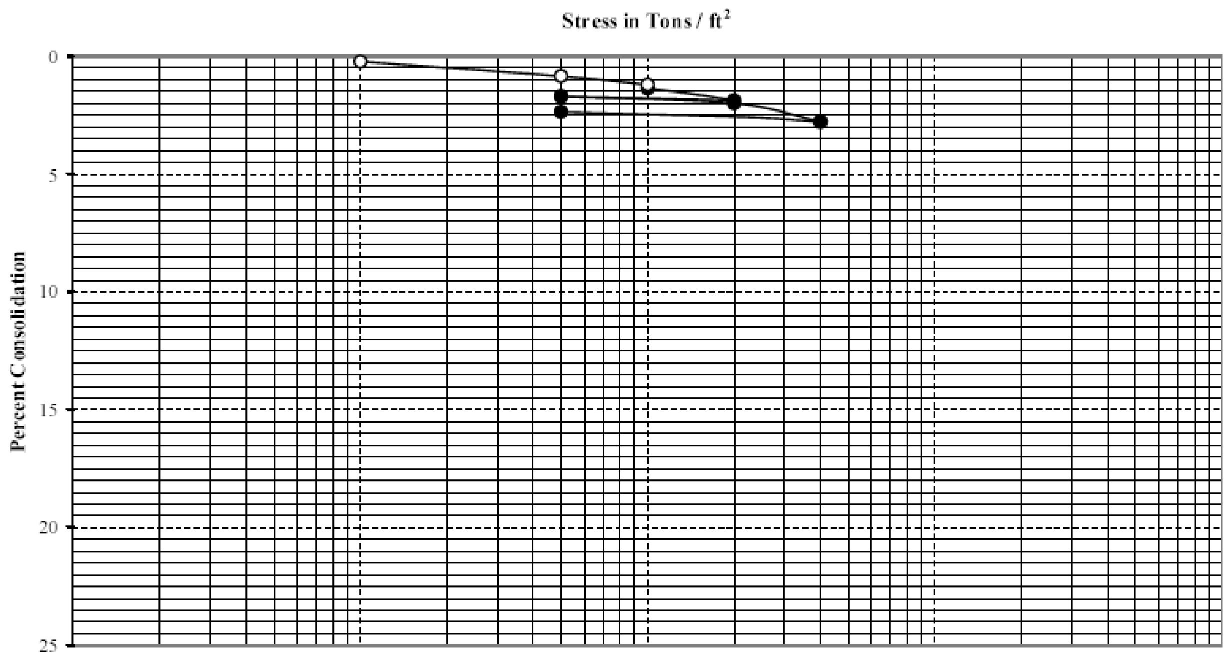


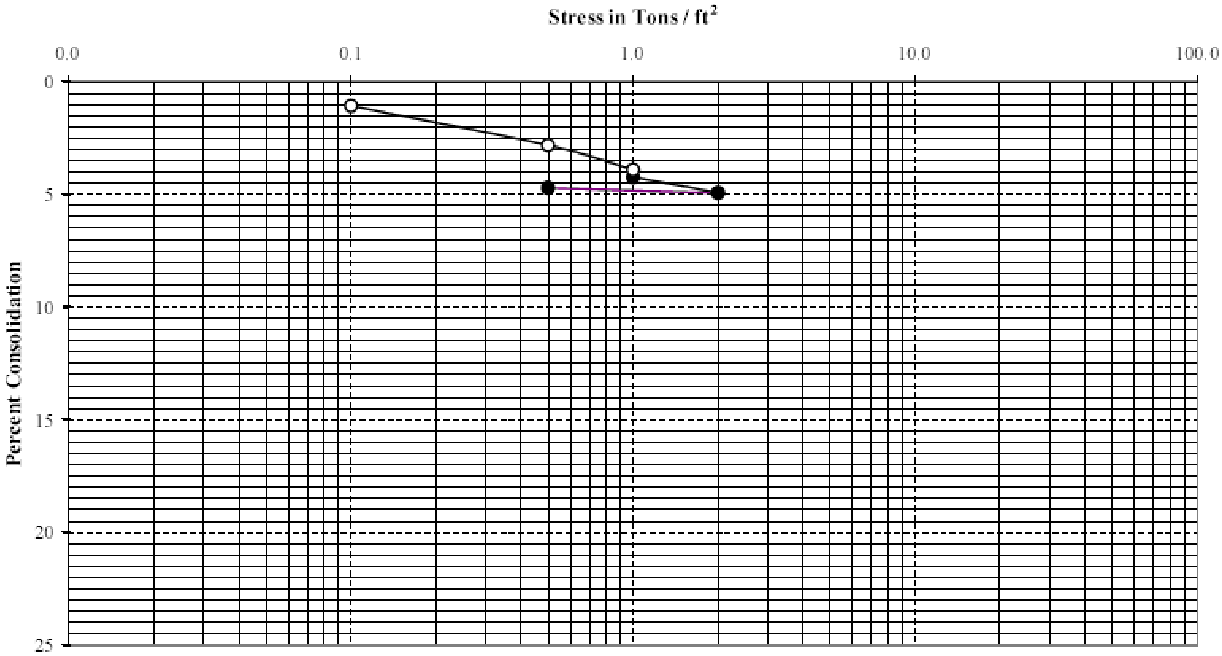
Plate CS - 6

C. Y. GEOTECH, INC. Geotechnical Engineering and Engineering Geology	Consolidation Test
Gilbert Salazar	
Date : 09-2007 / P.N. No: CYG-07-4948	

Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
B-2	46	4	18	1.0	2.4

Classification : Grayish brown gravelly sand (Qa)

Hydroconsolidation = 0.3 %



C. Y. GEOTECH, INC.

Geotechnical Engineering and Engineering Geology

Gilbert Salazar

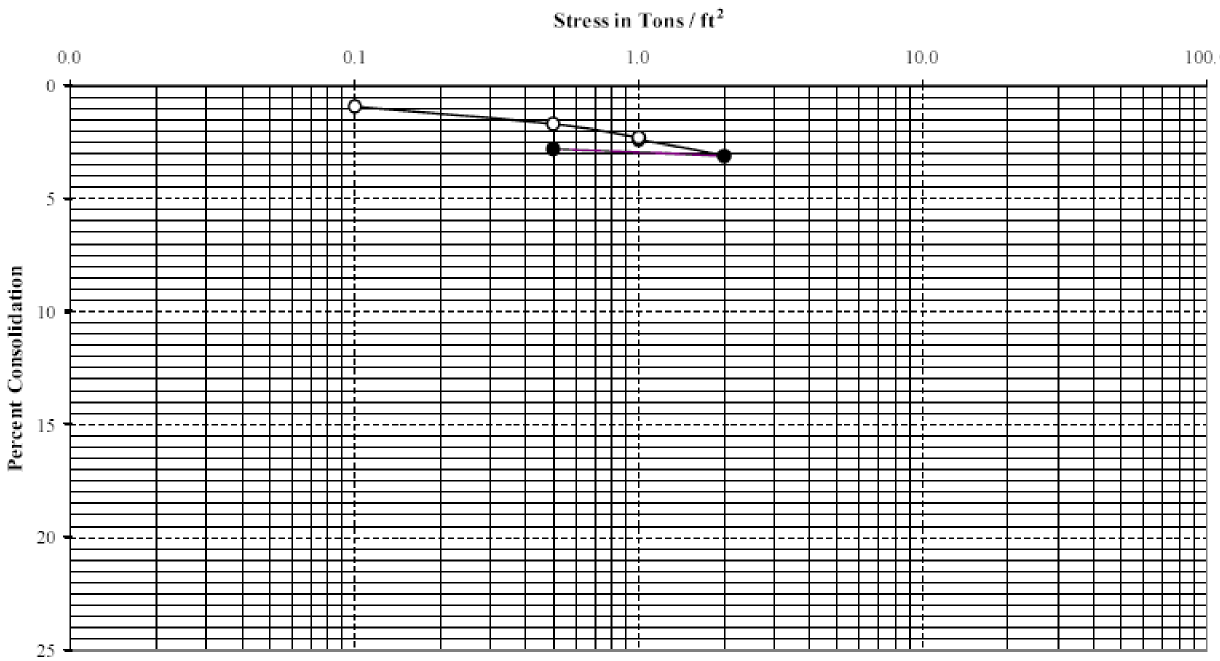
Date : 09-2007 | P.N. No: CYG-07-4948

Consolidation Test

Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
B-2	56	17	17	1.0	2.4

Classification : Brown clayey silt (Qa)

Hydroconsolidation = 0.1 %



C. Y. GEOTECH, INC.
Geotechnical Engineering and Engineering Geology

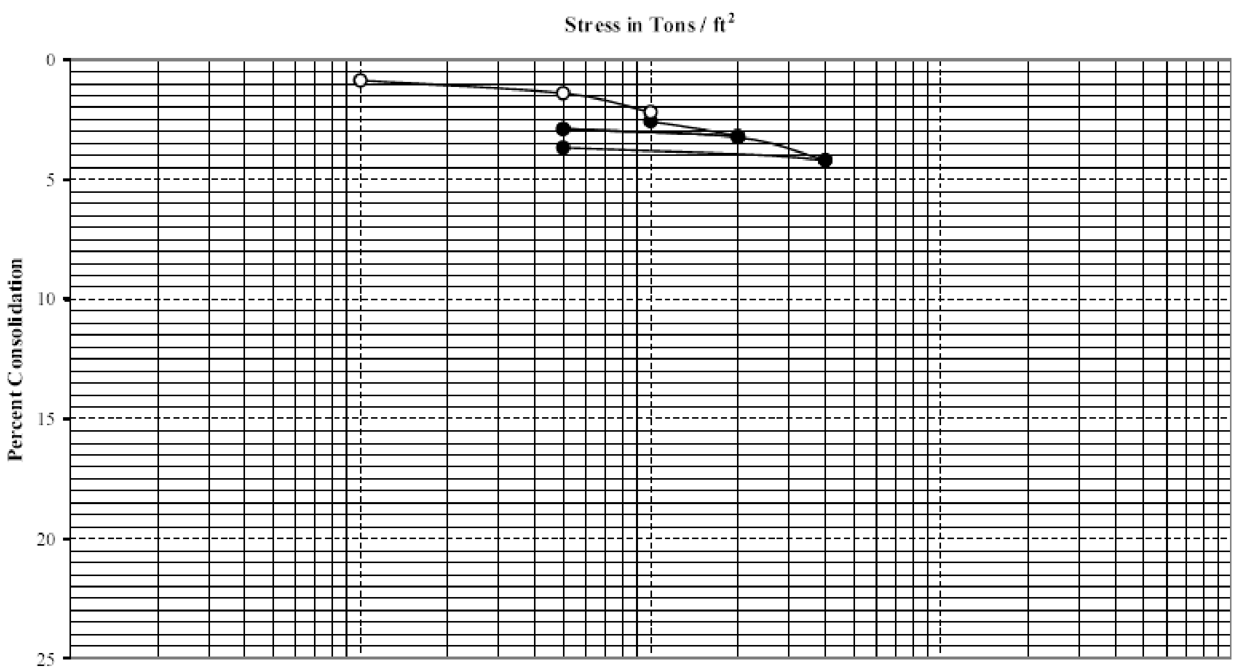
Gilbert Salazar

Date : 09-2007 | P.N. No: CYG-07-4948

Consolidation Test

Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
B-2	66	15	15	1.0	2.4

Classification : Brown sandy silty clay (Qa)
Hydroconsolidation = 0.4 %



C. Y. GEOTECH, INC. Geotechnical Engineering and Engineering Geology		Consolidation Test
Gilbert Salazar		
Date : 09-2007	P.N. No: CYG-07-4948	

Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
B-4	33	11	32	1.0	2.4

Classification : Brown clayey sandy silt (Qa)
Hydroconsolidation = 0.1 %

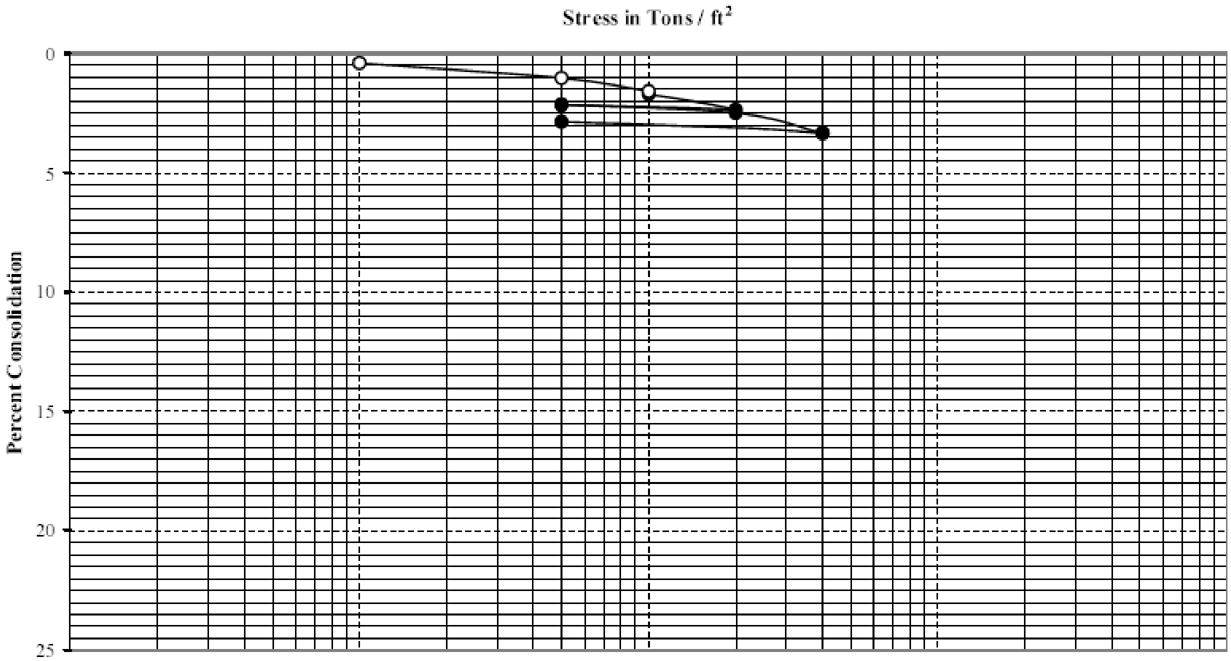
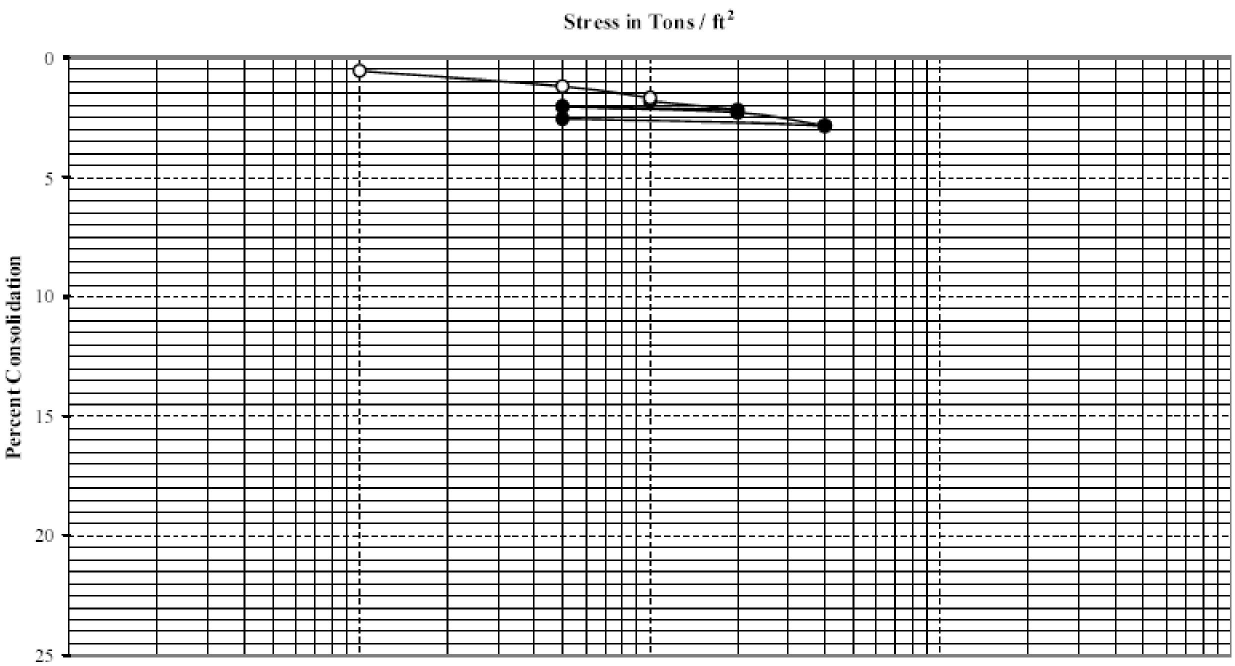


Plate CS - 10

C. Y. GEOTECH, INC. Geotechnical Engineering and Engineering Geology	Consolidation Test
Gilbert Salazar	
Date : 09-2007 / P.N. No: CYG-07-4948	Classification : Light brown gravelly sand (Qa) Hydroconsolidation = 0.1 %

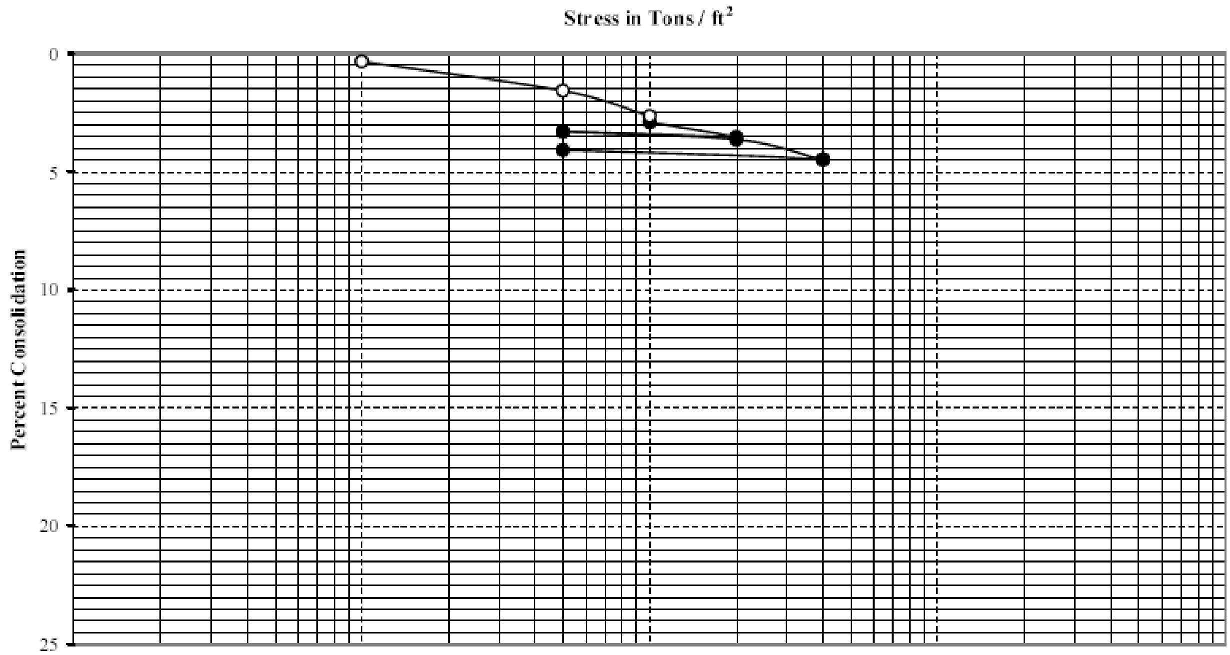
Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
B-4	43	3	11	1.0	2.4



C. Y. GEOTECH, INC. Geotechnical Engineering and Engineering Geology		**Consolidation Test**					
Gilbert Salazar		Boring	Depth (feet)	Water Content (%) Before	After	Height (inches)	Diameter (inches)
Date : 09-2007	P.N. No: CYG-07-4948	B-4	53	12	14	1.0	2.4

Classification : Light brown clayey silty sand (Qa)

Hydroconsolidation = 0.3 %



C. Y. GEOTECH, INC.

Geotechnical Engineering and Engineering Geology

Gilbert Salazar

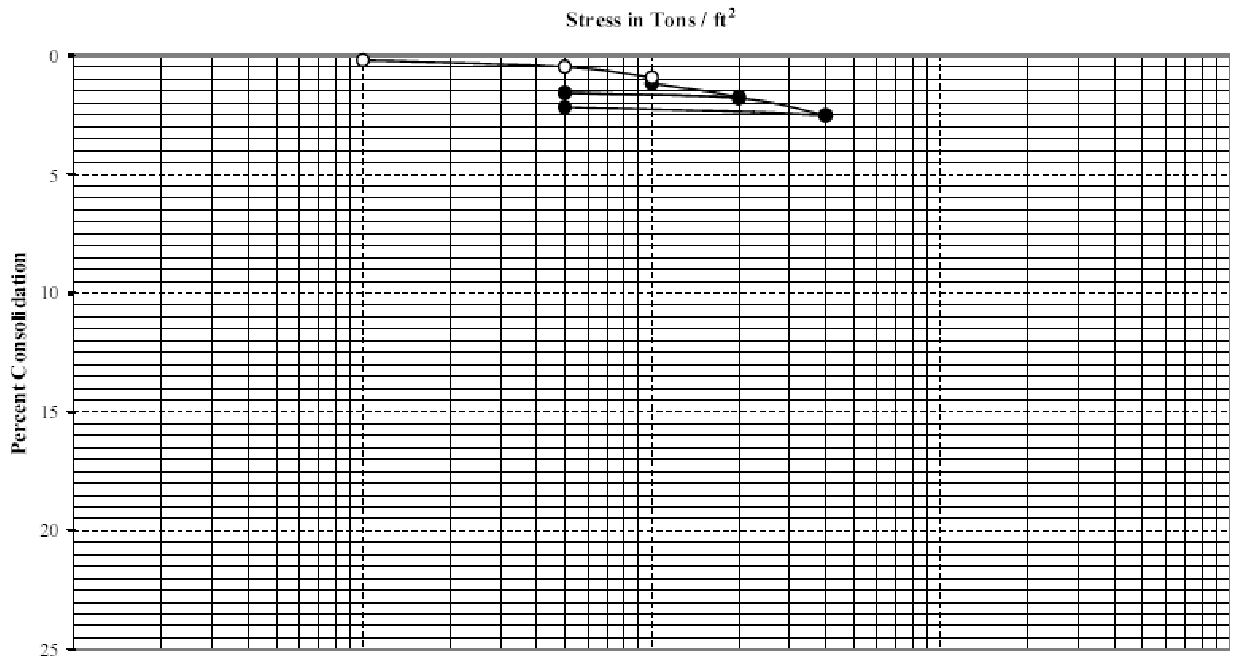
Date : 09-2007 | P.N. No: CYG-07-4948

Consolidation Test

Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
B-4	78	2	18	1.0	2.4

Classification : Light brown gravelly sand (Qa)

Hydroconsolidation = 0.3 %



C. Y. GEOTECH, INC.

Geotechnical Engineering and Engineering Geology

Gilbert Salazar

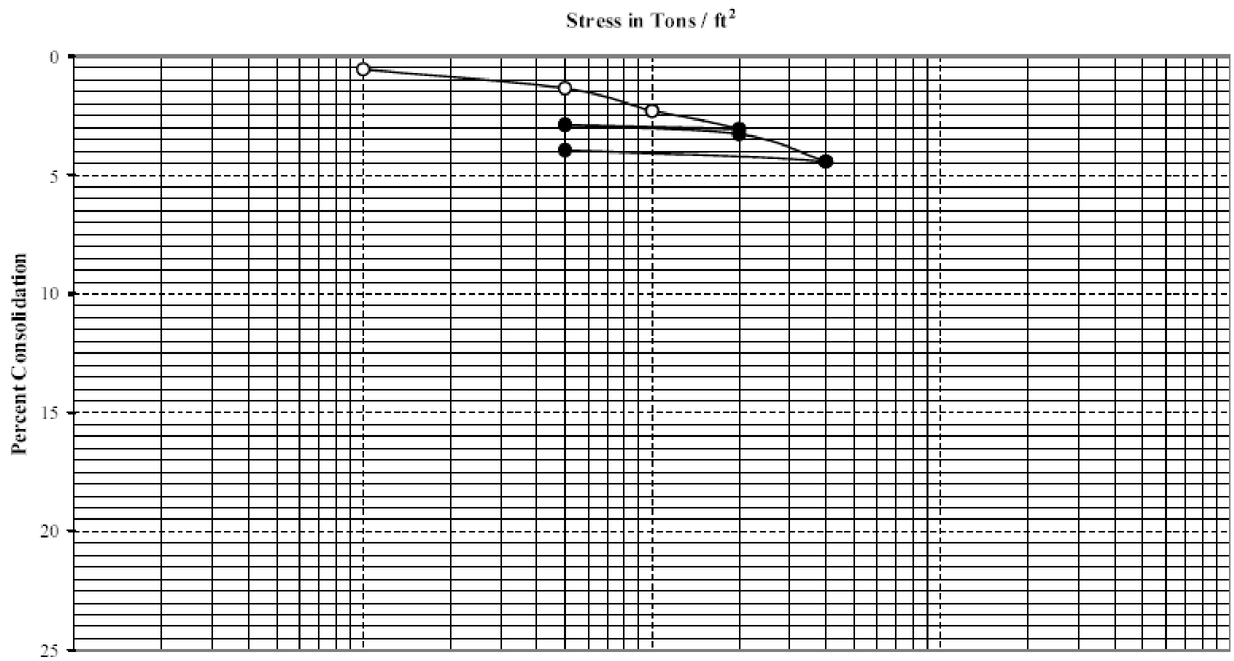
Date : 09-2007 | P.N. No: CYG-07-4948

Consolidation Test

Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
B-4	83	1	15	1.0	2.4

Classification : Light brown gravelly sand (Qa)

Hydroconsolidation = 0 %



C. Y. GEOTECH, INC.

Geotechnical Engineering and Engineering Geology

Gilbert Salazar

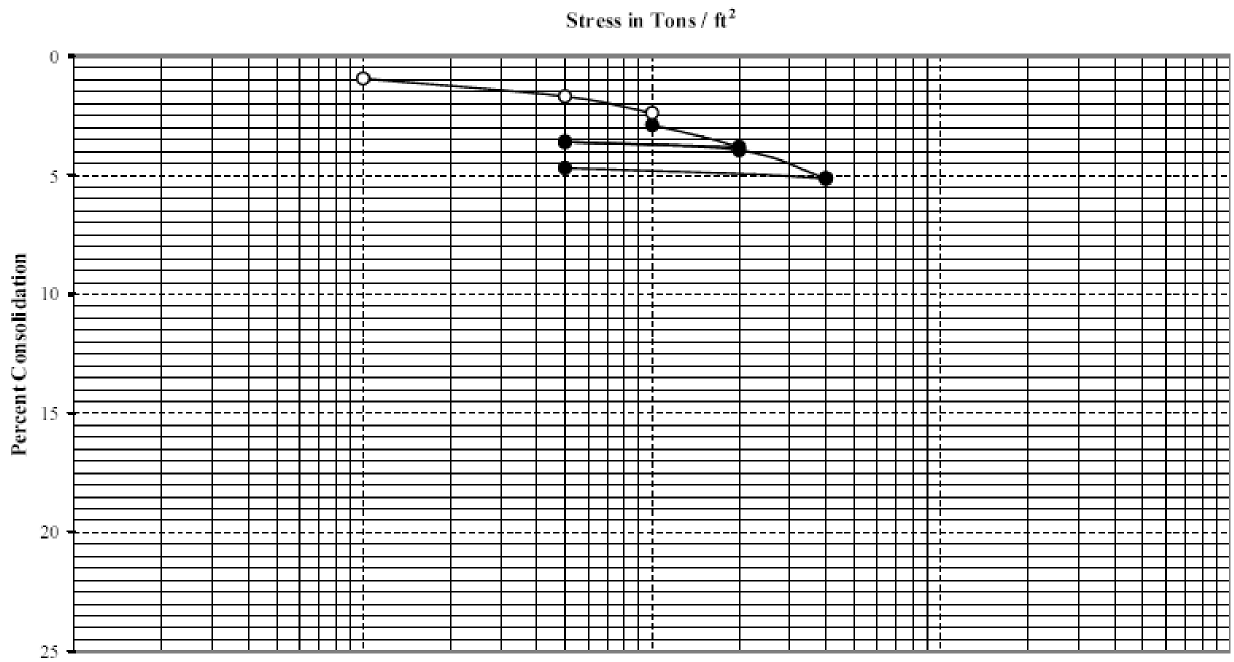
Date : 09-2007 | P.N. No: CYG-07-4948

Consolidation Test

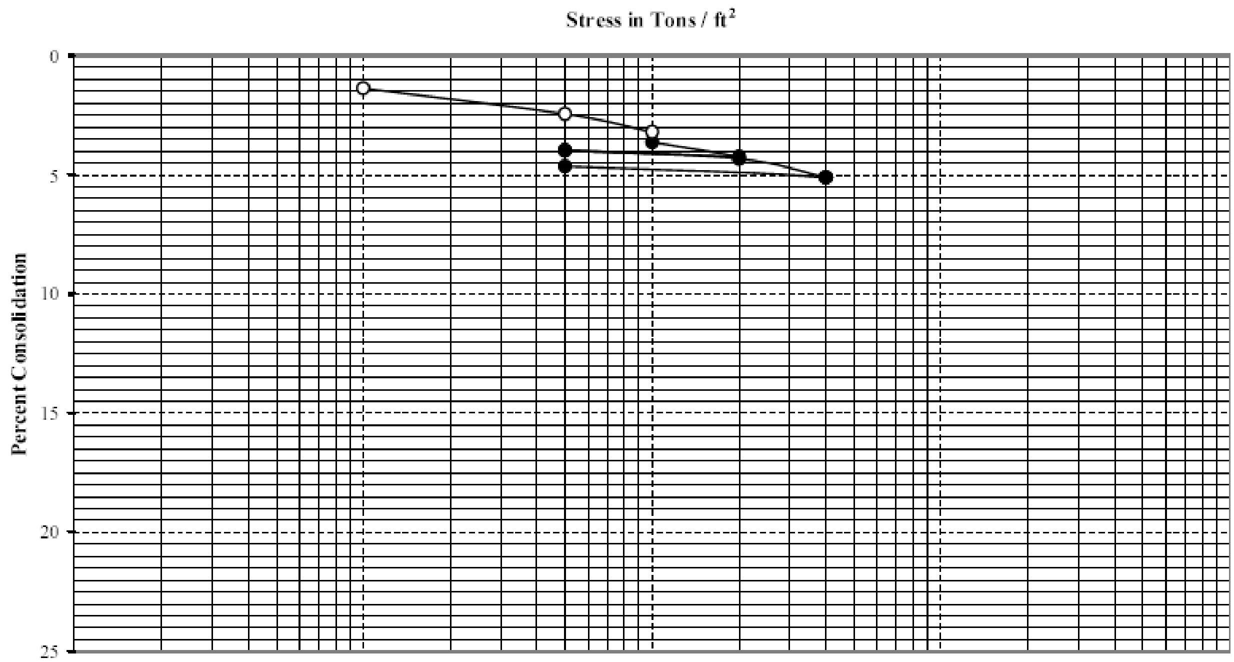
Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
B-5	34	9	19	1.0	2.4

Classification : Light brown sand (Qa)

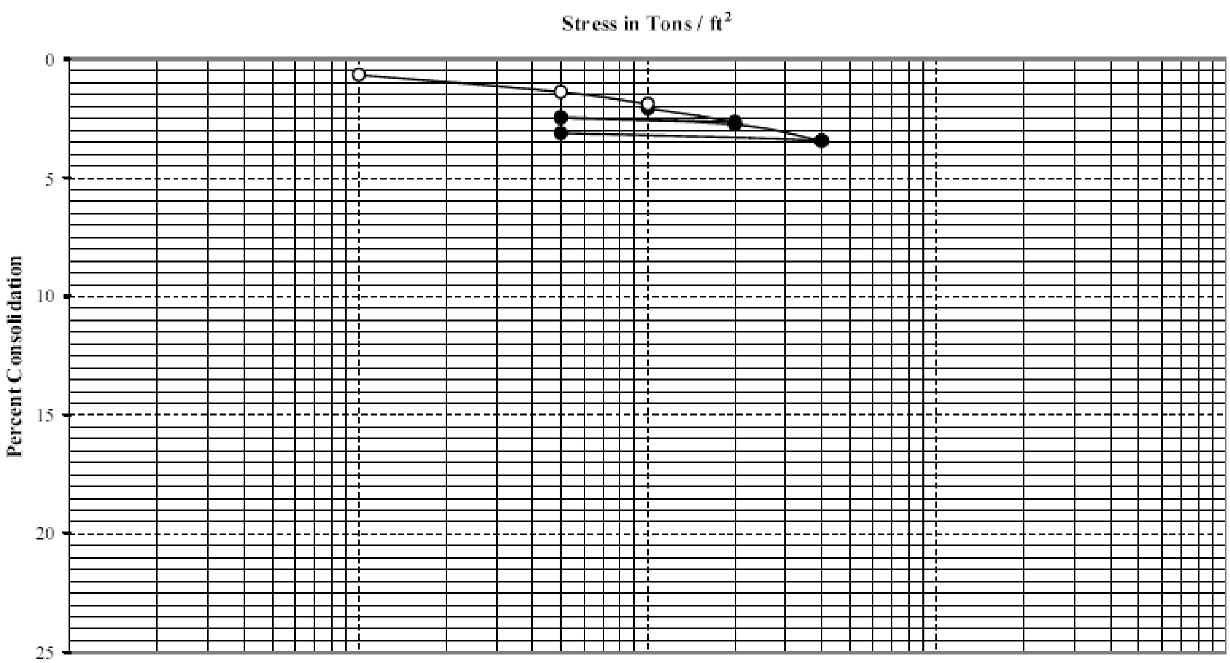
Hydroconsolidation = 0.5 %



C. Y. GEOTECH, INC. Geotechnical Engineering and Engineering Geology		Consolidation Test					
		Boring	Depth (feet)	Water Content (%) Before	After	Height (inches)	Diameter (inches)
Gilbert Salazar		B-5	44	19	19	1.0	2.4
Date : 09-2007	P.N. No: CYG-07-4948	Classification : Light brown clayey sand silt (Qa) Hydroconsolidation = 0.4 %					



C. Y. GEOTECH, INC. Geotechnical Engineering and Engineering Geology		**Consolidation Test**
Gilbert Salazar		Boring: B-5; Depth (feet): 54; Water Content (%) Before: 2, After: 19; Height (inches): 1.0; Diameter (inches): 2.4
Date : 09-2007	P.N. No: CYG-07-4948	Classification : Light brown sand (Qa) Hydroconsolidation = 0.2 %



C. Y. GEOTECH, INC.

Geotechnical Engineering and Engineering Geology

Gilbert Salazar

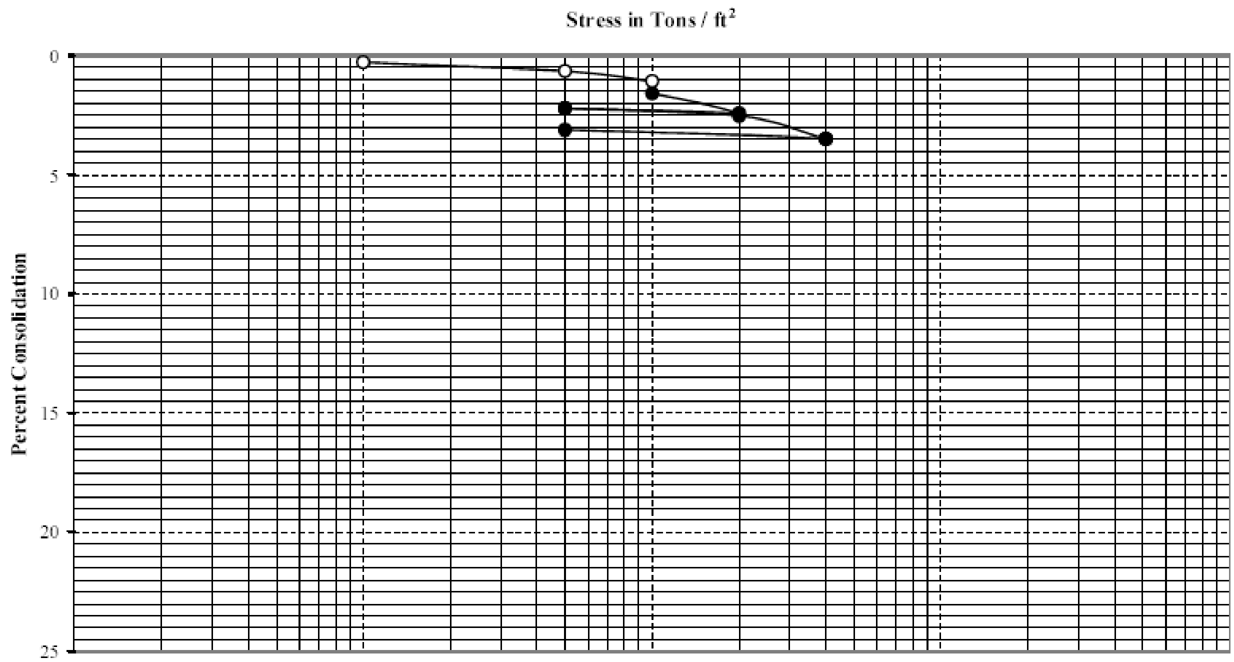
Date : 09-2007 | P.N. No: CYG-07-4948

Consolidation Test

Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
B-5	74	5	16	1.0	2.4

Classification : Light brown gravelly sand (Qa)

Hydroconsolidation = 0.5 %



C. Y. GEOTECH, INC.
Geotechnical Engineering and Engineering Geology

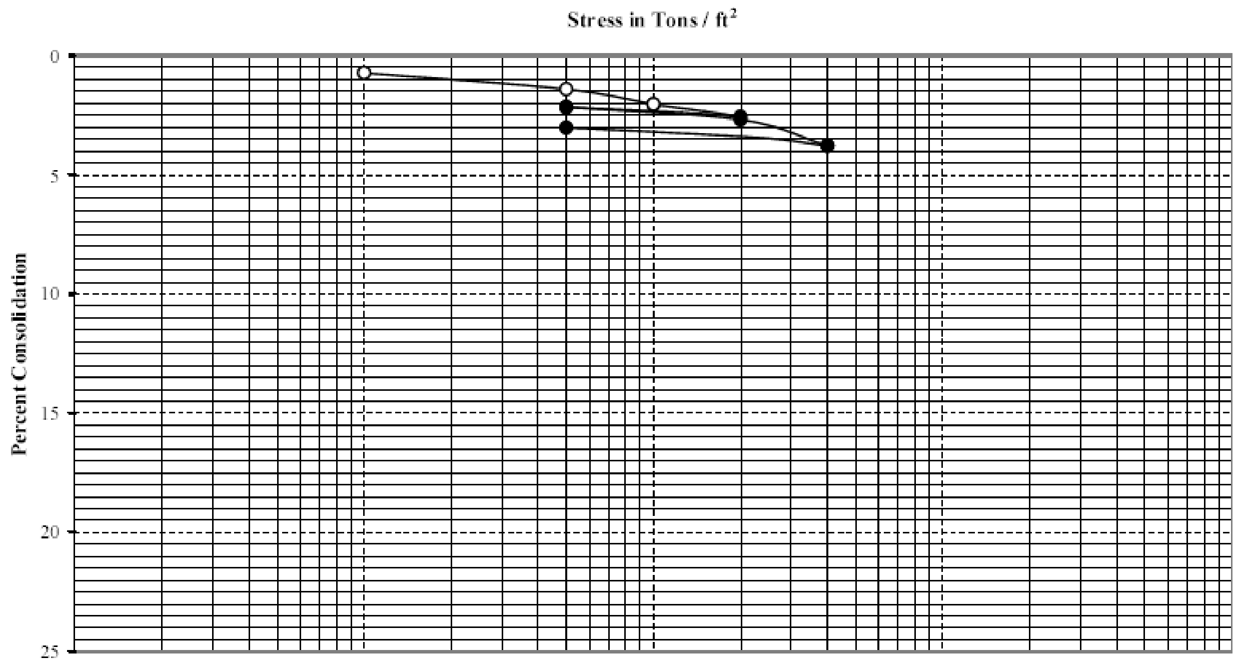
Gilbert Salazar

Date : 09-2007 | P.N. No: CYG-07-4948

Consolidation Test

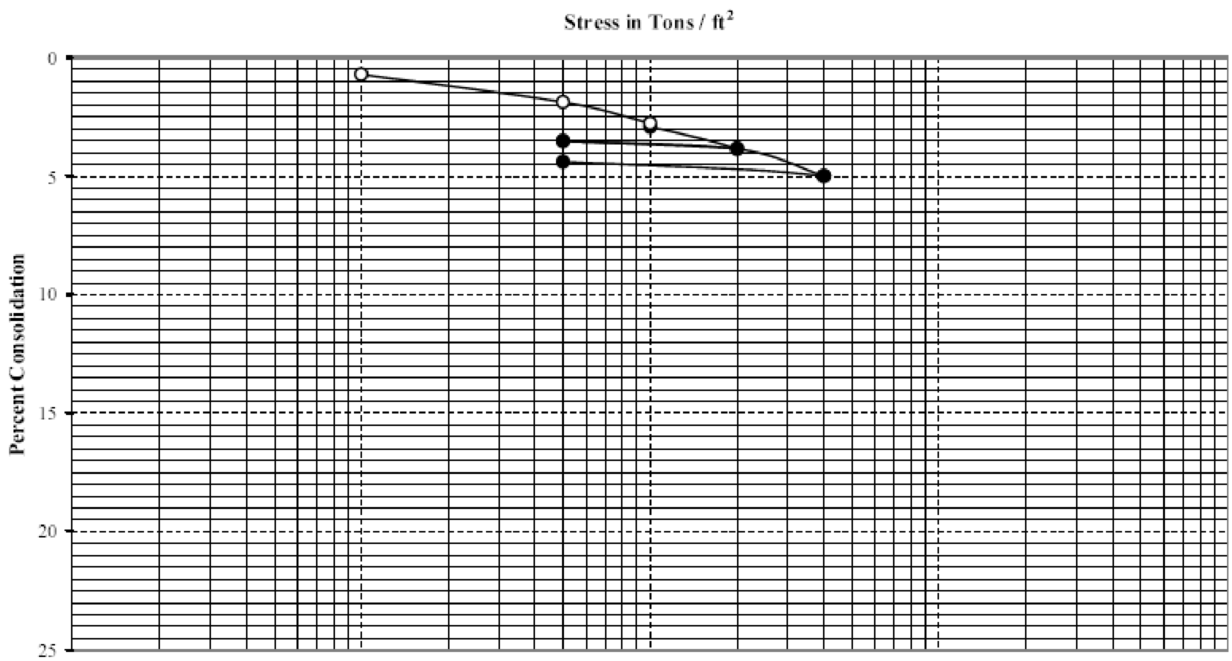
Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
B-7	29	17	21	1.0	2.4

Classification : Light brown clayey sandy silt (Qa)
Hydroconsolidation = 0 %



C. Y. GEOTECH, INC. Geotechnical Engineering and Engineering Geology	Consolidation Test
Gilbert Salazar	
Date : 09-2007 / P.N. No: CYG-07-4948	Classification : Light brown clayey sandy silt (Qa) Hydroconsolidation = 0.2 %

Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
B-7	34	16	19	1.0	2.4



C. Y. GEOTECH, INC. Geotechnical Engineering and Engineering Geology		Consolidation Test					
Gilbert Salazar		Boring	Depth (feet)	Water Content (%) Before	After	Height (inches)	Diameter (inches)
		B-7	39	4	19	1.0	2.4
Date : 09-2007	P.N. No: CYG-07-4948	Classification : Light brown clayey sandy silt (Qa) Hydroconsolidation = 0.3 %					

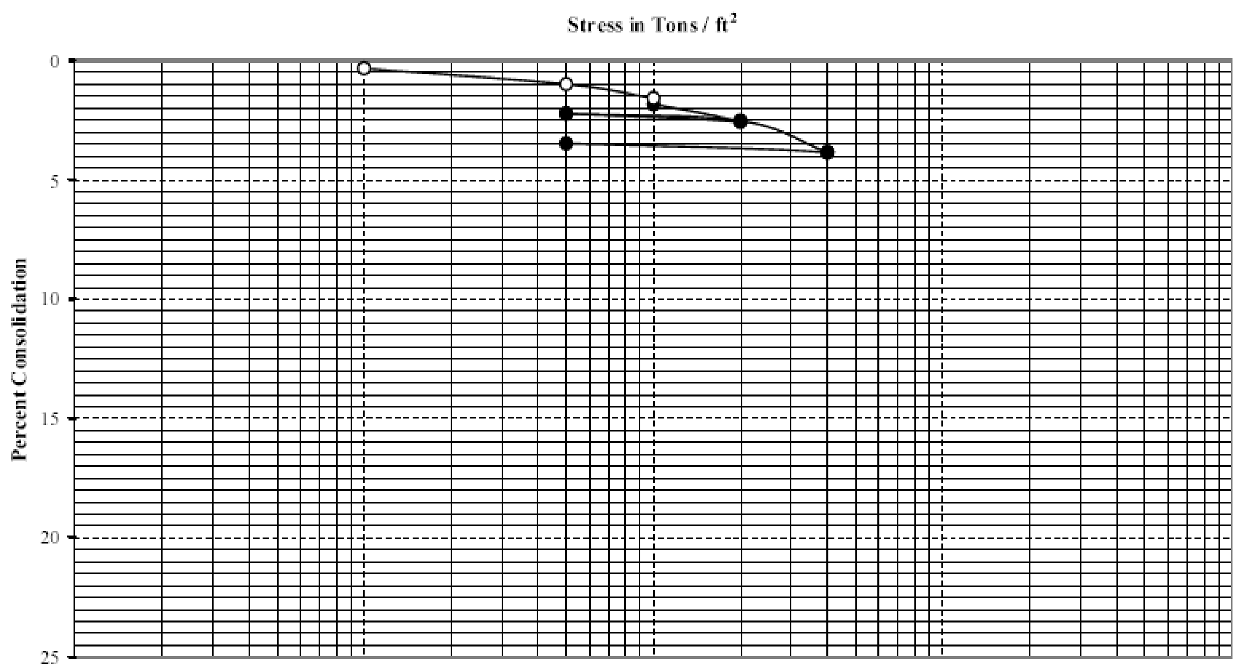


Plate CS - 21

C. Y. GEOTECH, INC.

Geotechnical Engineering and Engineering Geology

Gilbert Salazar

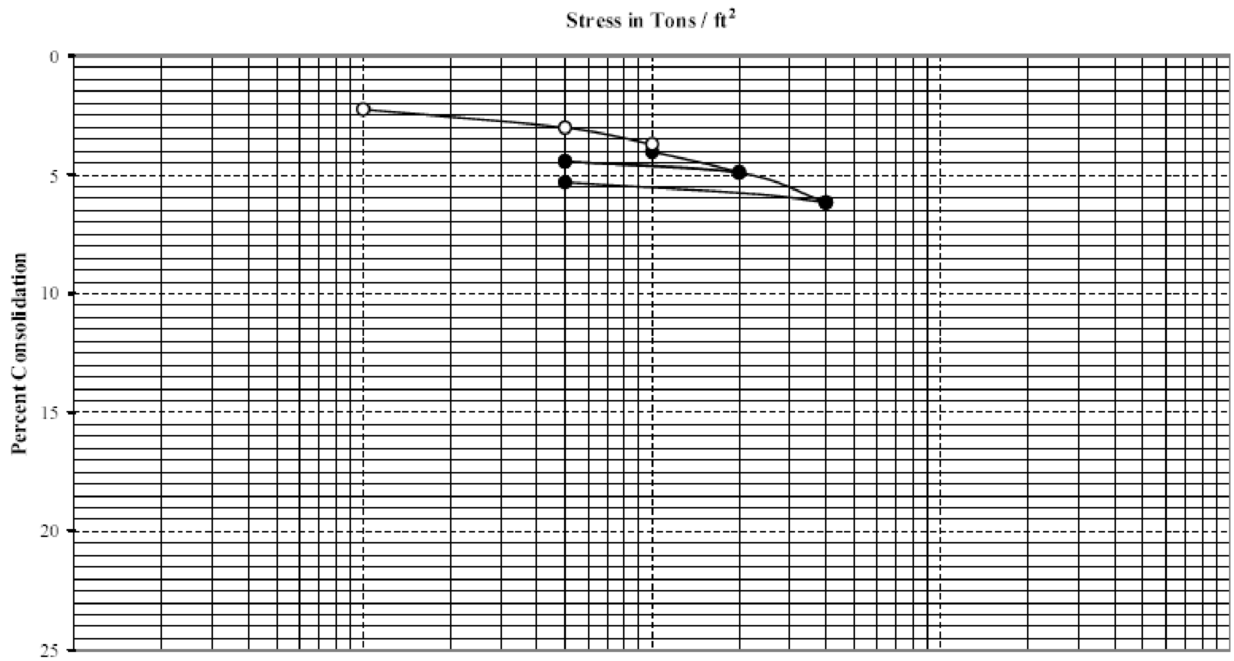
Date : 09-2007 | P.N. No: CYG-07-4948

Consolidation Test

Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
B-7	49	18	24	1.0	2.4

Classification : Light brown silty sand (Qa)

Hydroconsolidation = 0.3 %



C. Y. GEOTECH, INC.
Geotechnical Engineering and Engineering Geology

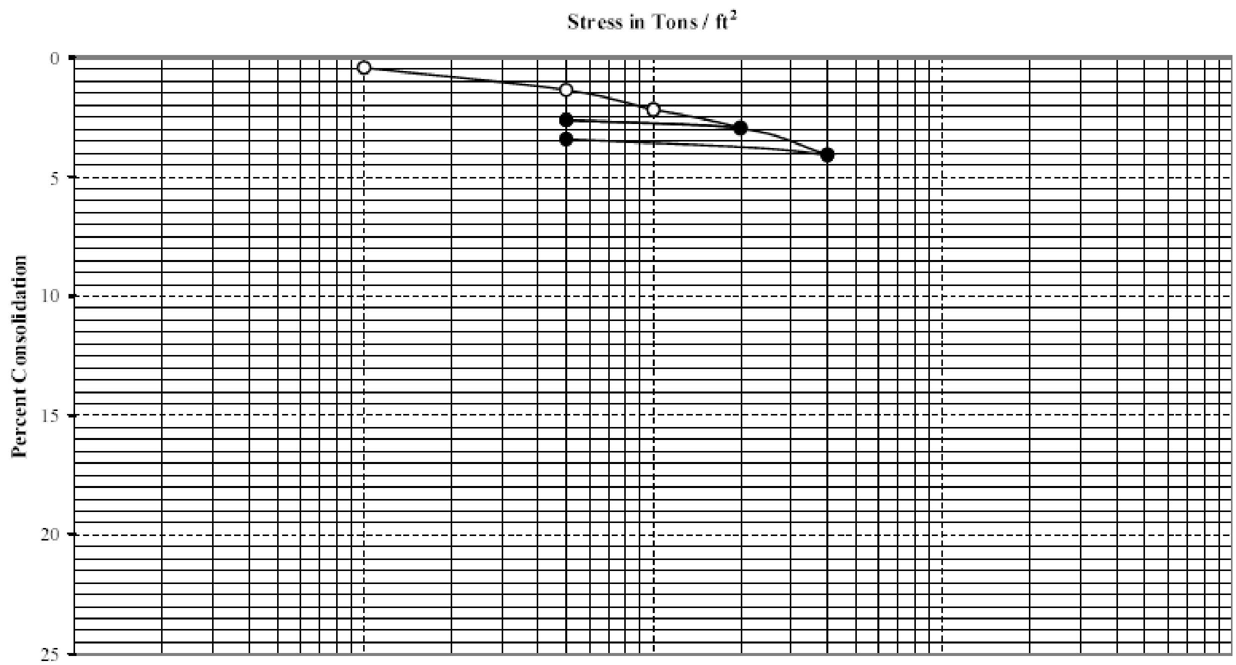
Gilbert Salazar

Date : 09-2007 | P.N. No: CYG-07-4948

Consolidation Test

Boring	Depth (feet)	Water Content (%) Before	After	Height (inches)	Diameter (inches)
B-7	59	18	22	1.0	2.4

Classification : Light brown silty sand (Qa)
Hydroconsolidation = 0 %



C. Y. GEOTECH, INC.

Geotechnical Engineering and Engineering Geology

Gilbert Salazar

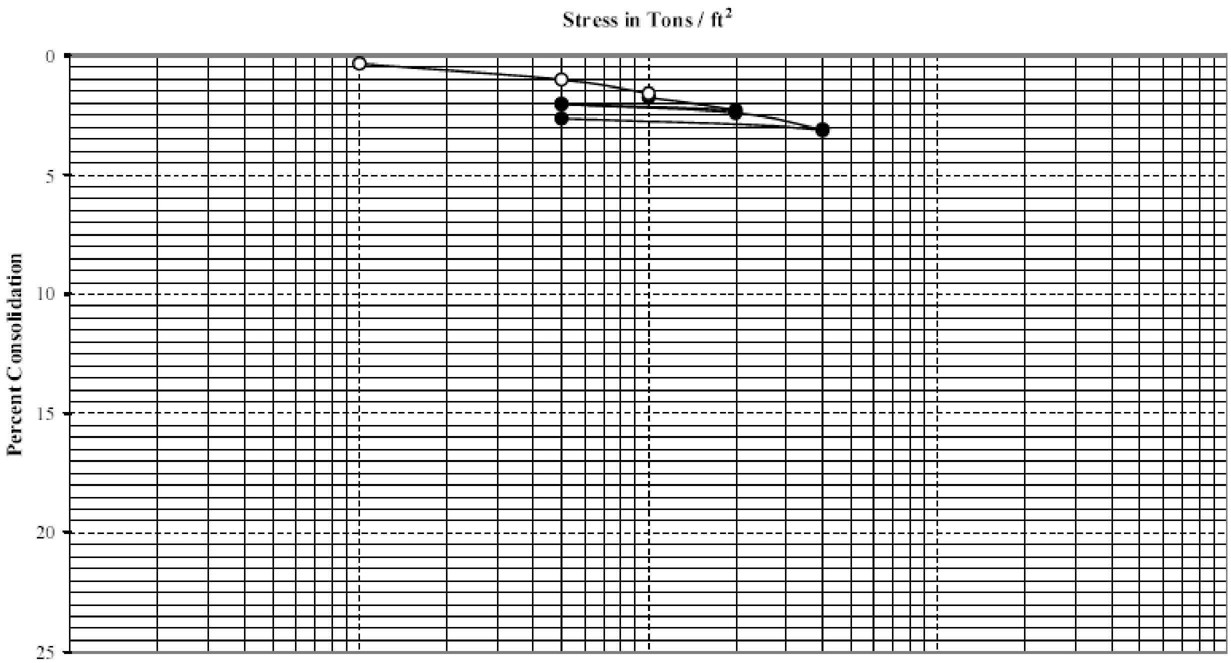
Date : 09-2007 | P.N. No: CYG-07-4948

Consolidation Test

Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
B-8	30	12	15	1.0	2.4

Classification : Brown clayey sandy silt (Qa)

Hydroconsolidation = 0.2 %



C. Y. GEOTECH, INC.
Geotechnical Engineering and Engineering Geology

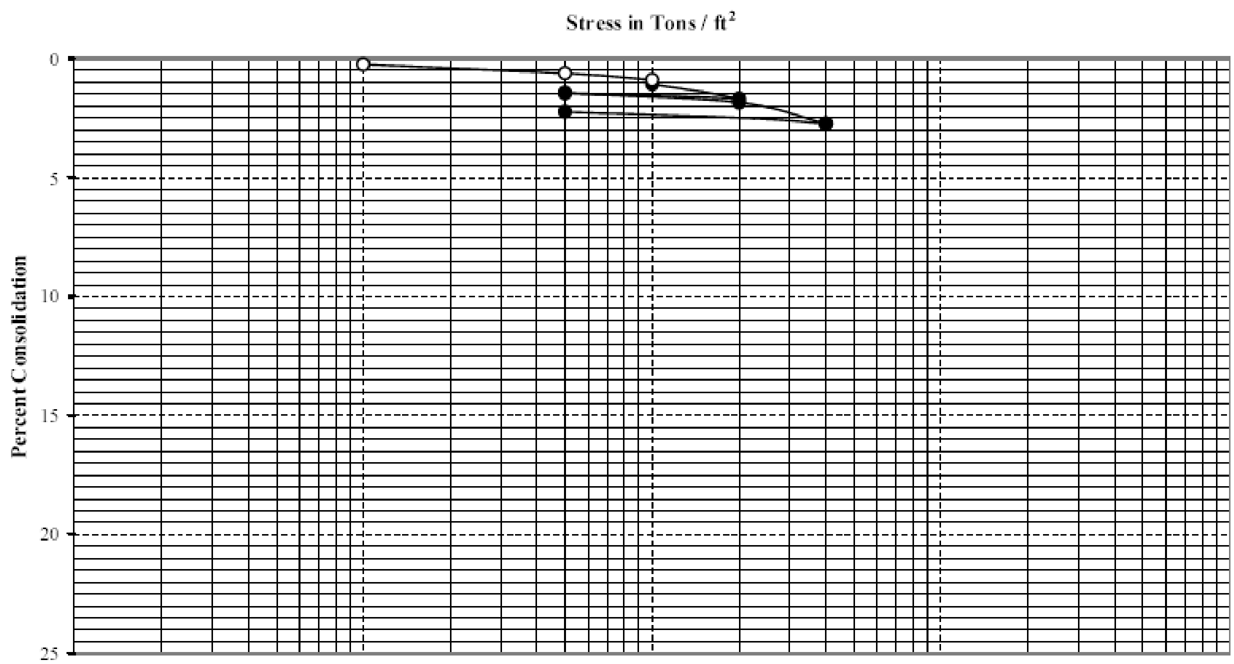
Gilbert Salazar

Date : 09-2007 | P.N. No: CYG-07-4948

Consolidation Test

Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
B-8	40	2	11	1.0	2.4

Classification : Light brown gravelly sand (Qa)
Hydroconsolidation = 0.2 %



C. Y. GEOTECH, INC.

Geotechnical Engineering and Engineering Geology

Gilbert Salazar

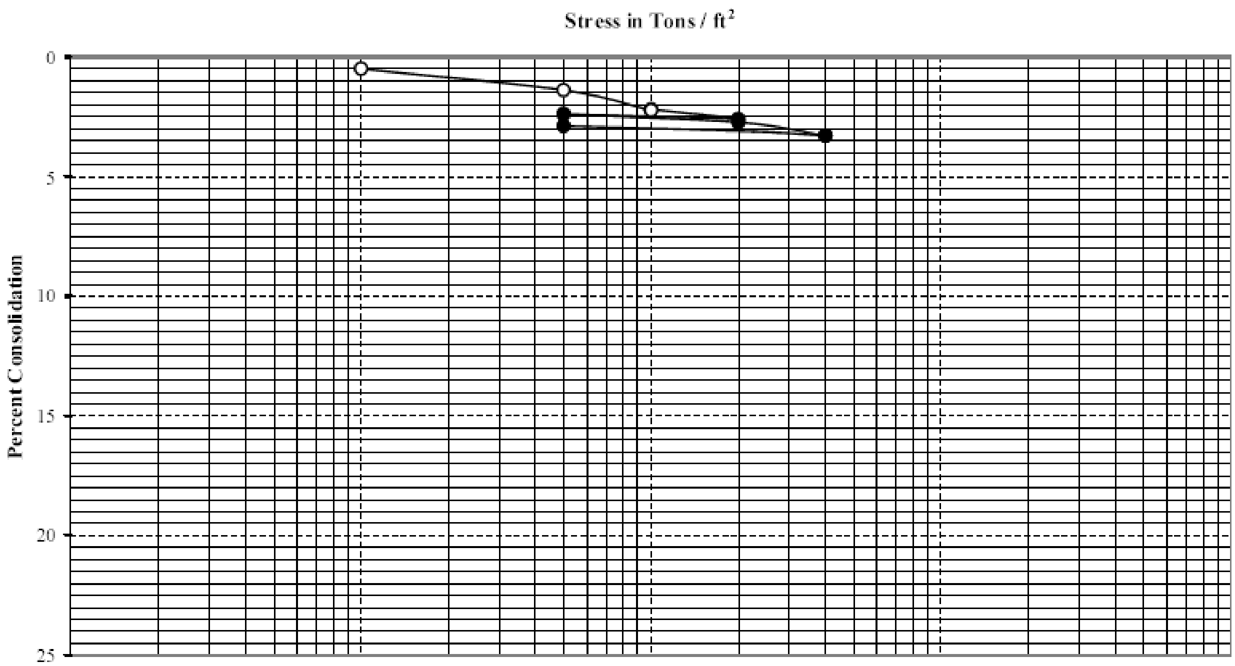
Date : 09-2007 | P.N. No: CYG-07-4948

Consolidation Test

Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
B-8	50	3	12	1.0	2.4

Classification : Light brown gravelly sand (Qa)

Hydroconsolidation = 0 %



C. Y. GEOTECH, INC.

Geotechnical Engineering and Engineering Geology

Gilbert Salazar

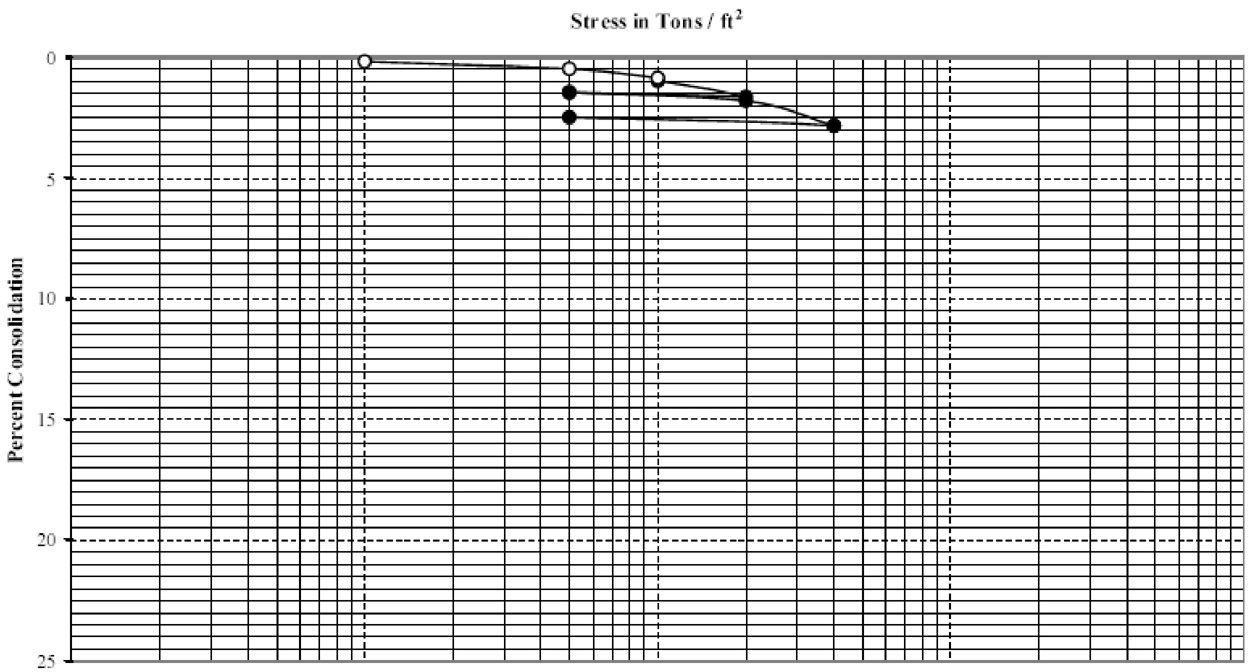
Date : 09-2007	P.N. No: CYG-07-4948

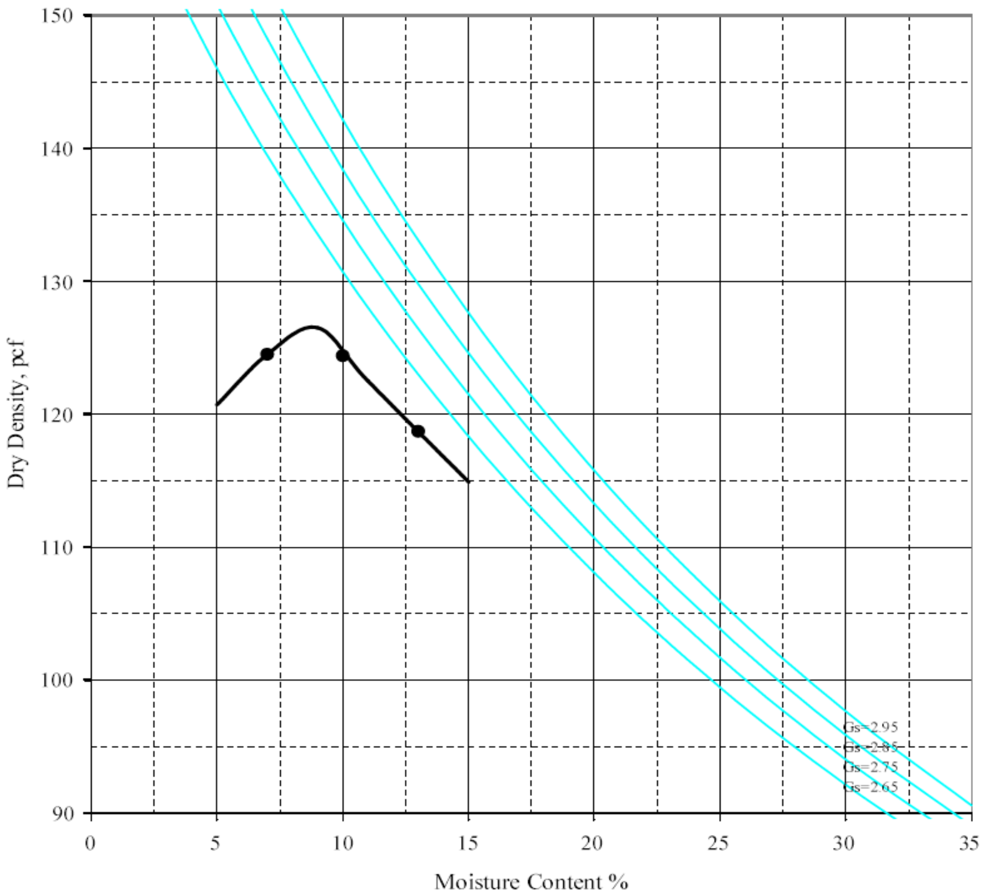
Consolidation Test

Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
B-8	60	6	13	1.0	2.4

Classification : Light brown silty sand (Qa)

Hydroconsolidation = 0.1 %





Maximum Dry Density = 126.5 pcf

Optimum Moisture Content = 9 %

Boring : B-1
Depth : 1 ~ 7'
Description : Grayish brown silty sand
(Qa)

C. Y. GEOTECH, INC.

Geotechnical Engineering
and Engineering Geology

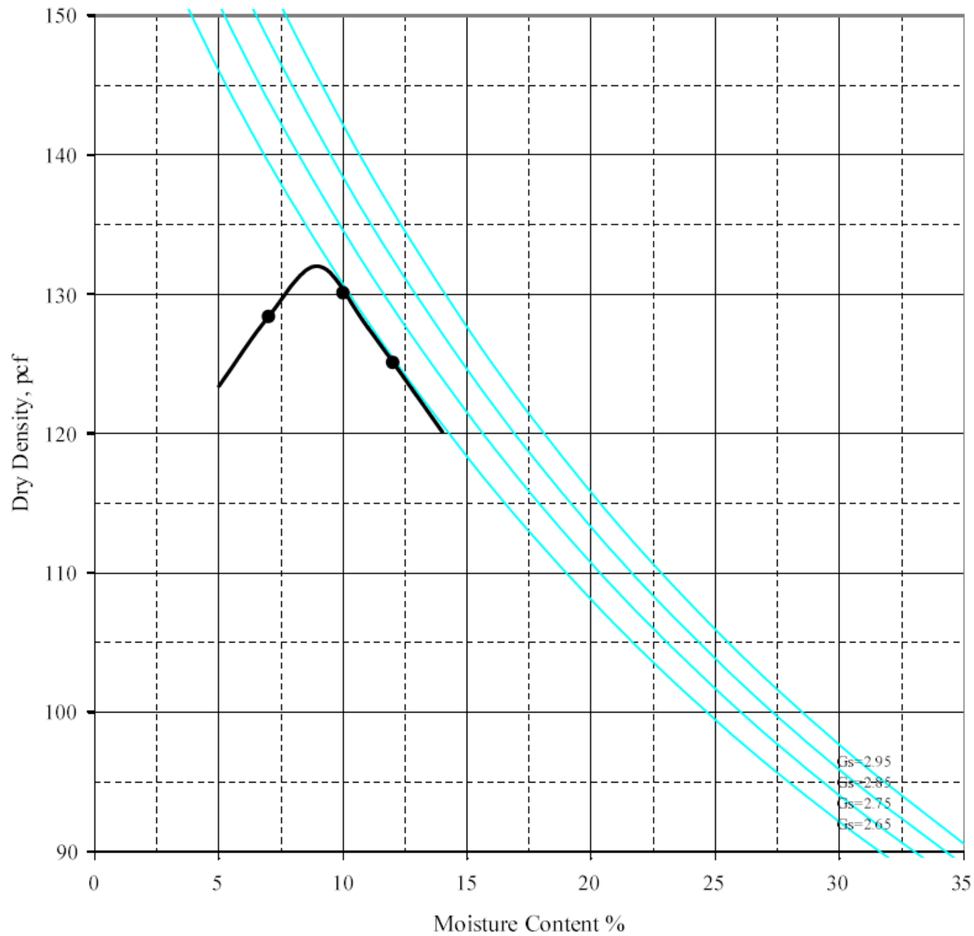
Gilbert Salazar

Date : 09-2007

P.N. No.: CYG-07-4948

Compaction Curve

Plate CM - 1



Maximum Dry Density = 132 pcf

Optimum Moisture Content = 9 %

Boring : B-4
Depth : 28' ~ 38'
Description : Light brown clayey sandy silt
(Qa)

C. Y. GEOTECH, INC.

Geotechnical Engineering
and Engineering Geology

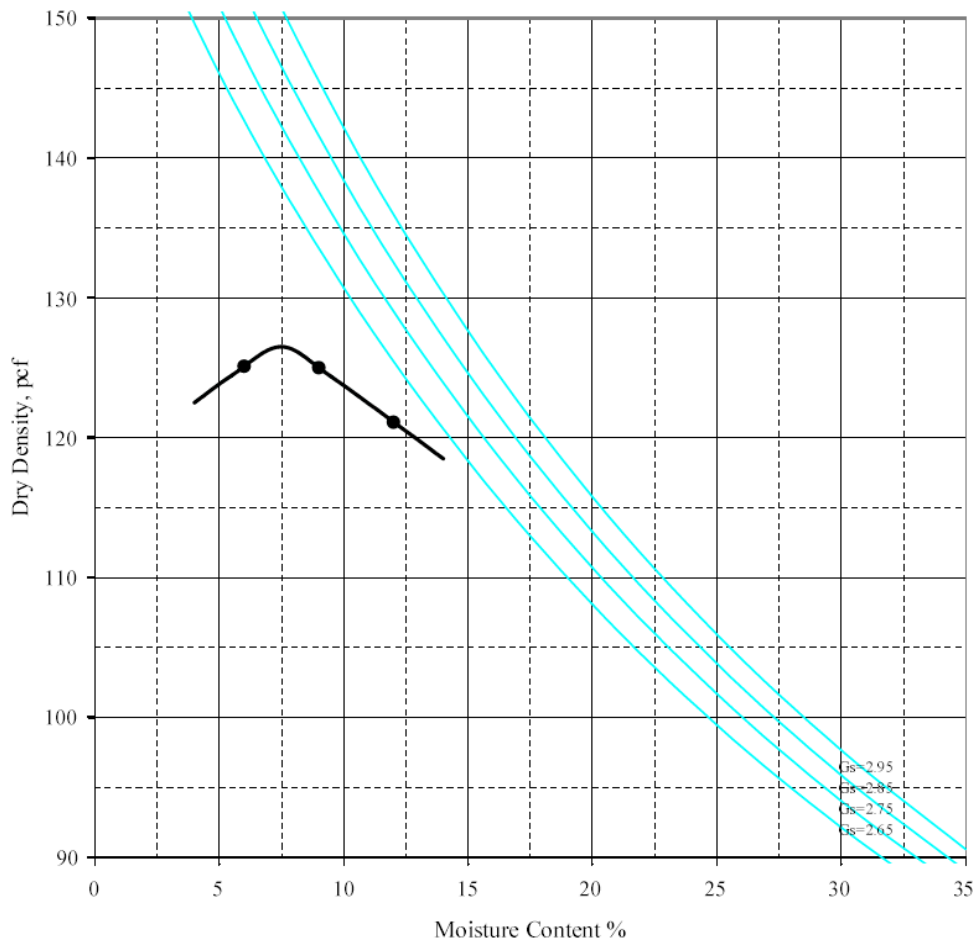
Gilbert Salazar

Date : 09-2007

P.N. No.: CYG-07-4948

Compaction Curve

Plate CM - 2



Maximum Dry Density = 126.5 pcf Optimum Moisture Content = 7.5 %	Boring : B-7 Depth : 1' ~ 10' Description : Light brown gravelly sand (Qa)
C. Y. GEOTECH, INC. Geotechnical Engineering and Engineering Geology	Gilbert Salazar Date : 09-2007 / P.N. No.: CYG-07-4948 **Compaction Curve**

Plate CM - 3

C. Y. GEOTECH, INC.

Geotechnical Engineering and Engineering Geology

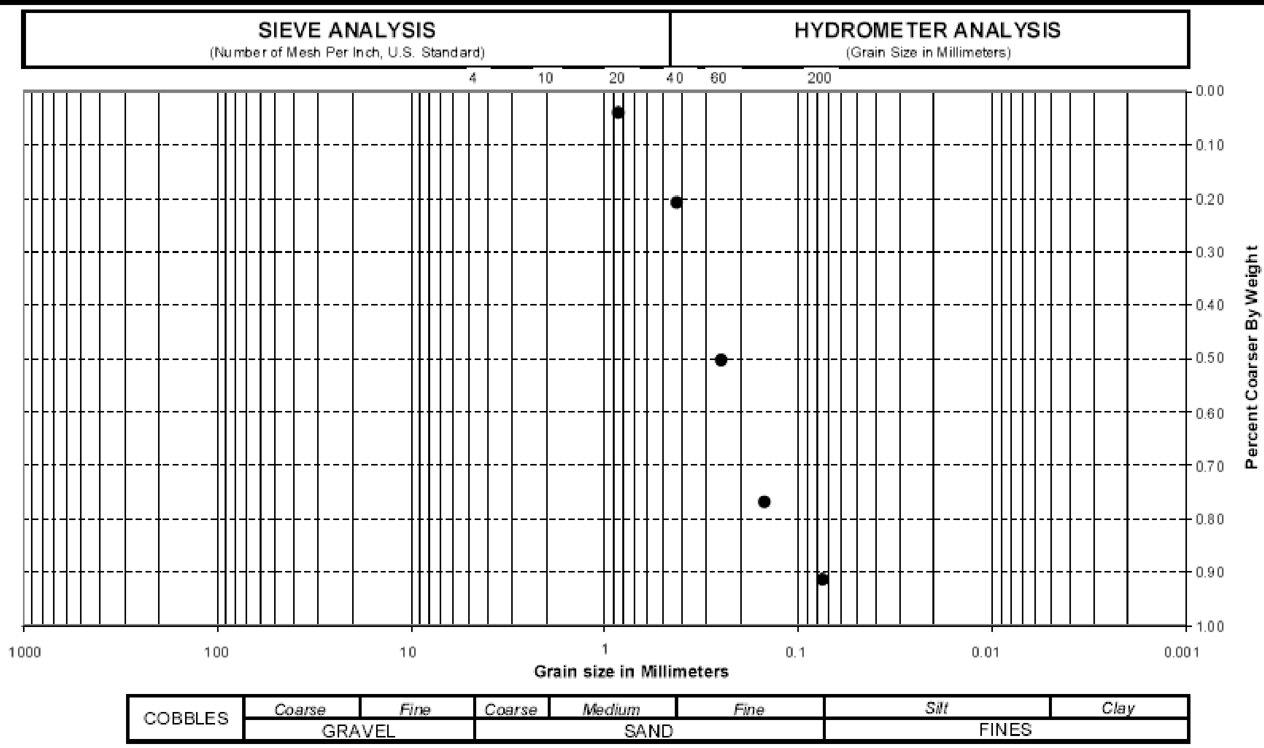
Gilbert Salazar

Date : 09-2007 | P.N. No.: CYG-07-4948

Sieve Analysis

Boring	Depth (feet)	γ Field (pcf)	γ Saturate (pcf)	Fines (%)	D_{50} (mm)
B-1	5	120	124	9	0.252

Classification : Grayish brown sand (Qa)



C. Y. GEOTECH, INC.

Geotechnical Engineering and Engineering Geology

Gilbert Salazar

Date : 06-2007 | P.N. No.: CYG-07-4948

Sieve / Hydrometer Analysis

Boring	Depth (feet)	γ Field (pcf)	γ Saturate (pcf)	Fines (%)	D_{50} (mm)
B-1	10	109	129	44	0.103

Classification : Grayish brown silty sand (Qa)
Clay Content = 10 %

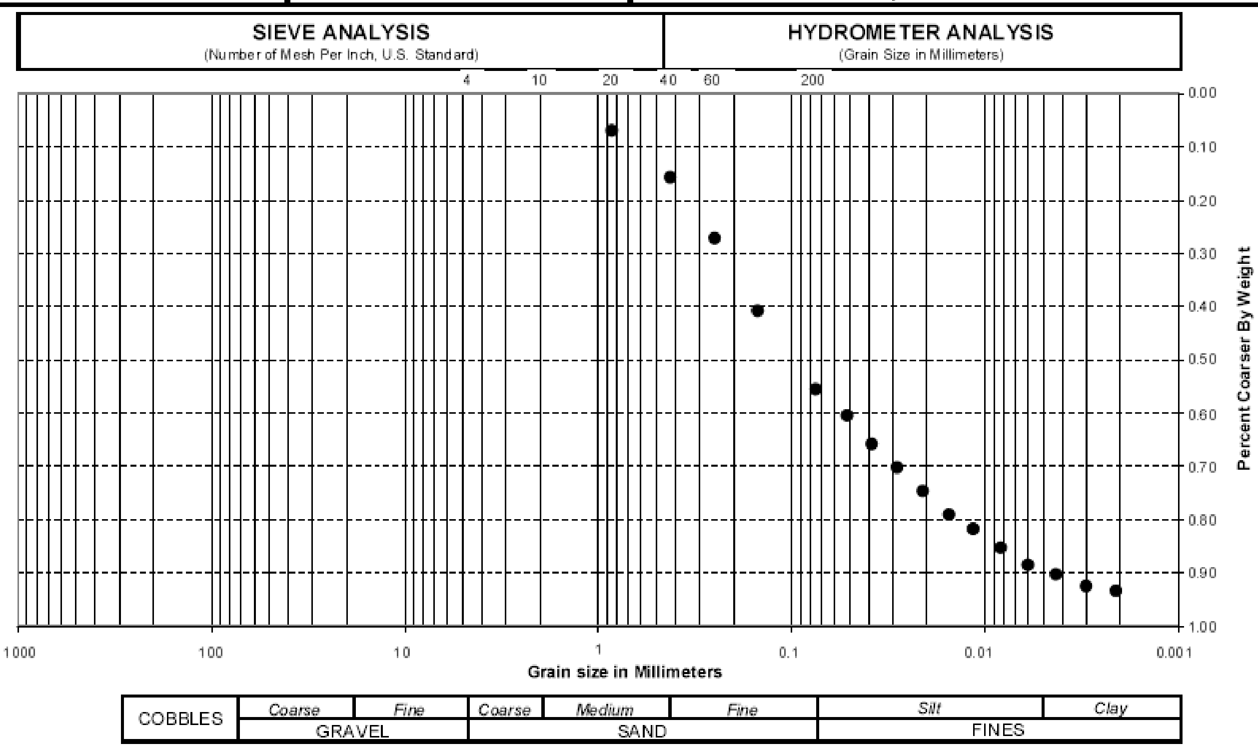


Plate GD - 2

C. Y. GEOTECH, INC.

Geotechnical Engineering and Engineering Geology

Gilbert Salazar

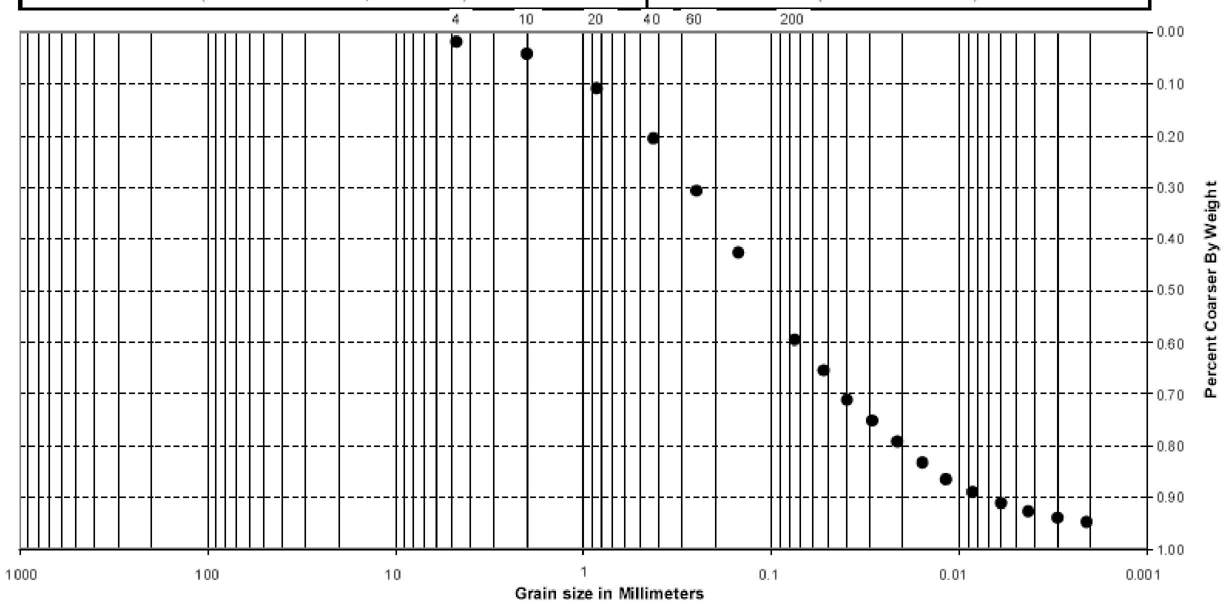
Date : 06-2007 | P.N. No.: CYG-07-4948

Sieve / Hydrometer Analysis

Boring	Depth (feet)	γ_{Field} (pcf)	$\gamma_{Saturate}$ (pcf)	Fines (%)	D_{50} (mm)
B-1	20	108	126	41	0.117

Classification : Grayish brown silty sand (Qa)
Clay Content = 8 %

SIEVE ANALYSIS (Number of Mesh Per Inch, U.S. Standard)	HYDROMETER ANALYSIS (Grain Size in Millimeters)



COBBLES	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
	GRAVEL		SAND			FINES	

Plate GD - 3

C. Y. GEOTECH, INC.

Geotechnical Engineering and Engineering Geology

Gilbert Salazar

Date : 06-2007 | P.N. No.: CYG-07-4948

Sieve / Hydrometer Analysis

Boring	Depth (feet)	γ_{Field} (pcf)	$\gamma_{Saturate}$ (pcf)	Fines (%)	D_{50} (mm)
B-1	25	120	134	41	0.138

Classification : Brown silty sand (Qa)
Clay Content = 8 %

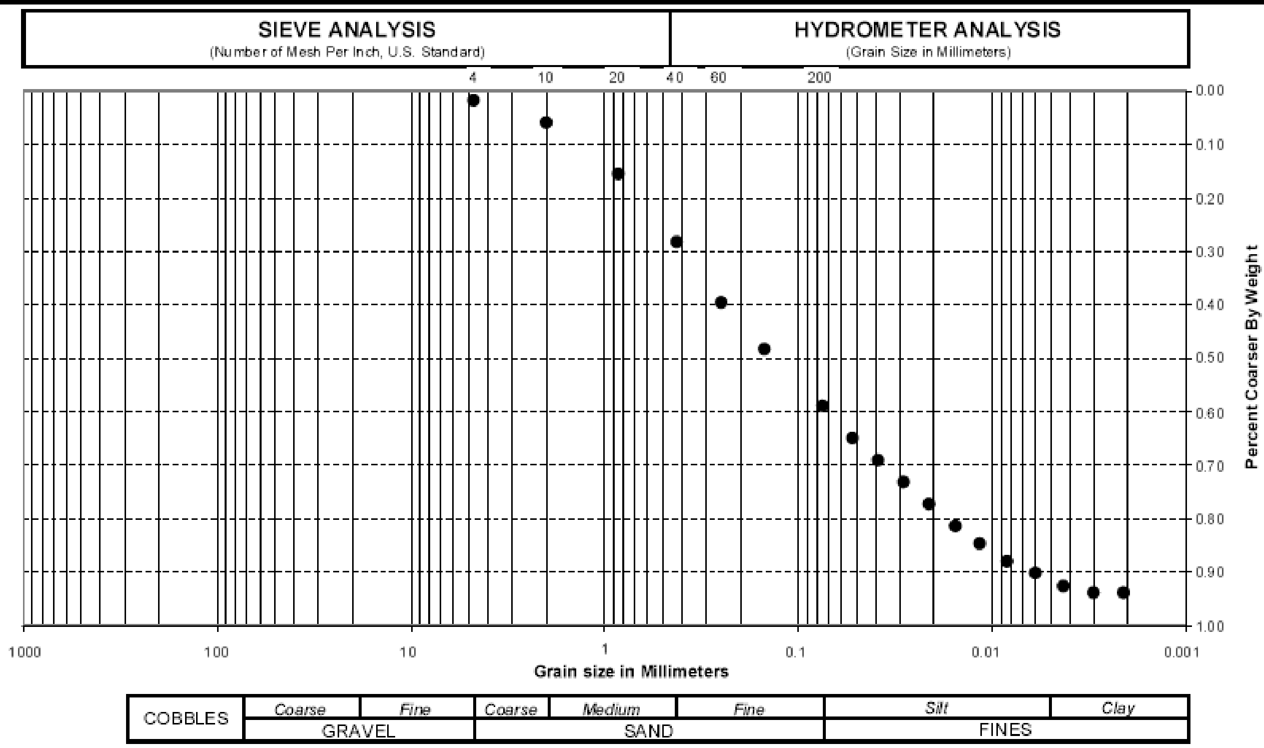


Plate GD - 4

C. Y. GEOTECH, INC.

Geotechnical Engineering and Engineering Geology

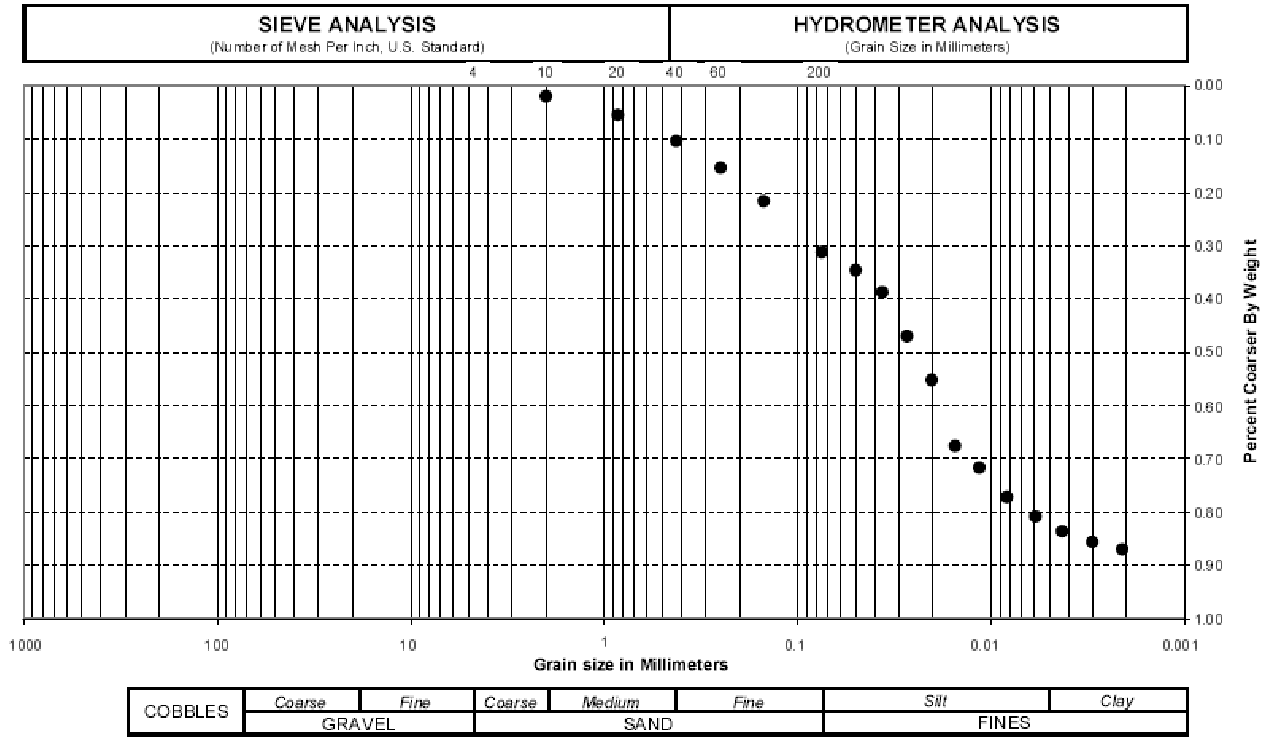
Gilbert Salazar

Date : 06-2007 | P.N. No. : CYG-07-4948

Sieve / Hydrometer Analysis

Boring	Depth (feet)	γ_{Field} (pcf)	$\gamma_{Saturate}$ (pcf)	Fines (%)	D_{50} (mm)
B-1	35	137	137	69	0.025

Classification : Brown clayey sandy silt (Qa)
Clay Content = 18 %



C. Y. GEOTECH, INC.
Geotechnical Engineering and Engineering Geology

Gilbert Salazar

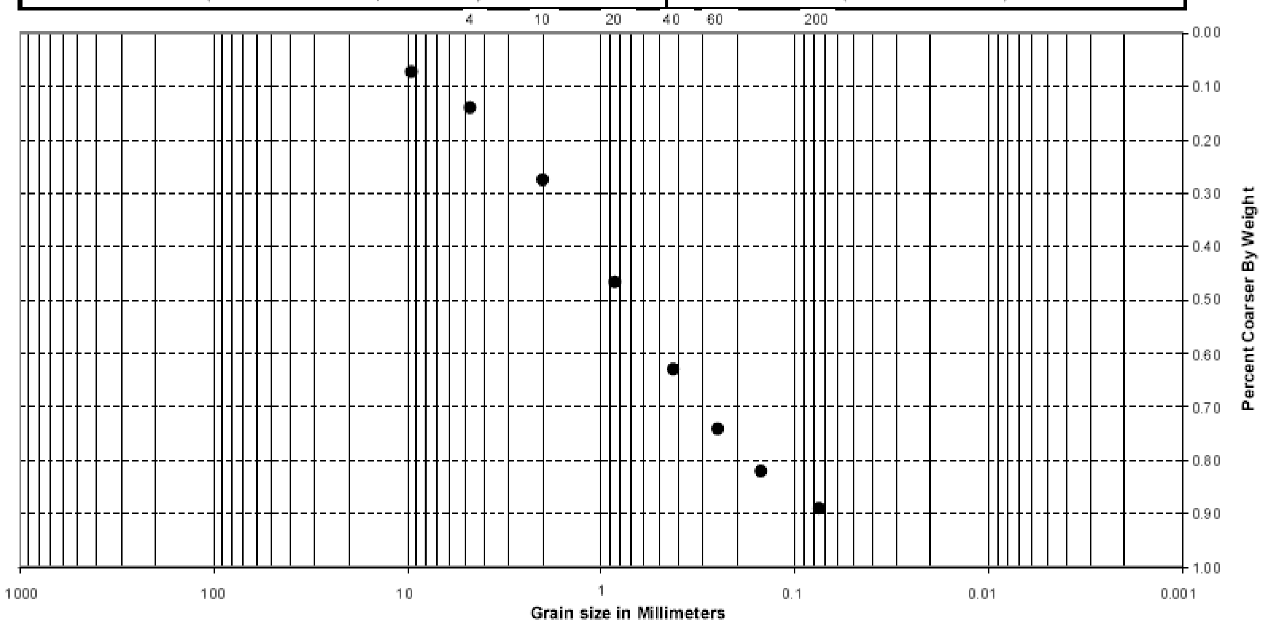
Date : 06-2007 | P.N. No.: CYG-07-4948

Sieve Analysis

Boring	Depth (feet)	γ Field (pcf)	γ Saturate (pcf)	Fines (%)	D_{50} (mm)
B-1	40	147	148	11	0.762

Classification : Brown gravelly sand (Qa)

SIEVE ANALYSIS (Number of Mesh Per Inch, U.S. Standard)	HYDROMETER ANALYSIS (Grain Size in Millimeters)



COBBLES	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
	GRAVEL		SAND			FINES	

Plate GD - 6

C. Y. GEOTECH, INC.

Geotechnical Engineering and Engineering Geology

Gilbert Salazar

Date : 06-2007 | P.N. No.: CYG-07-4948

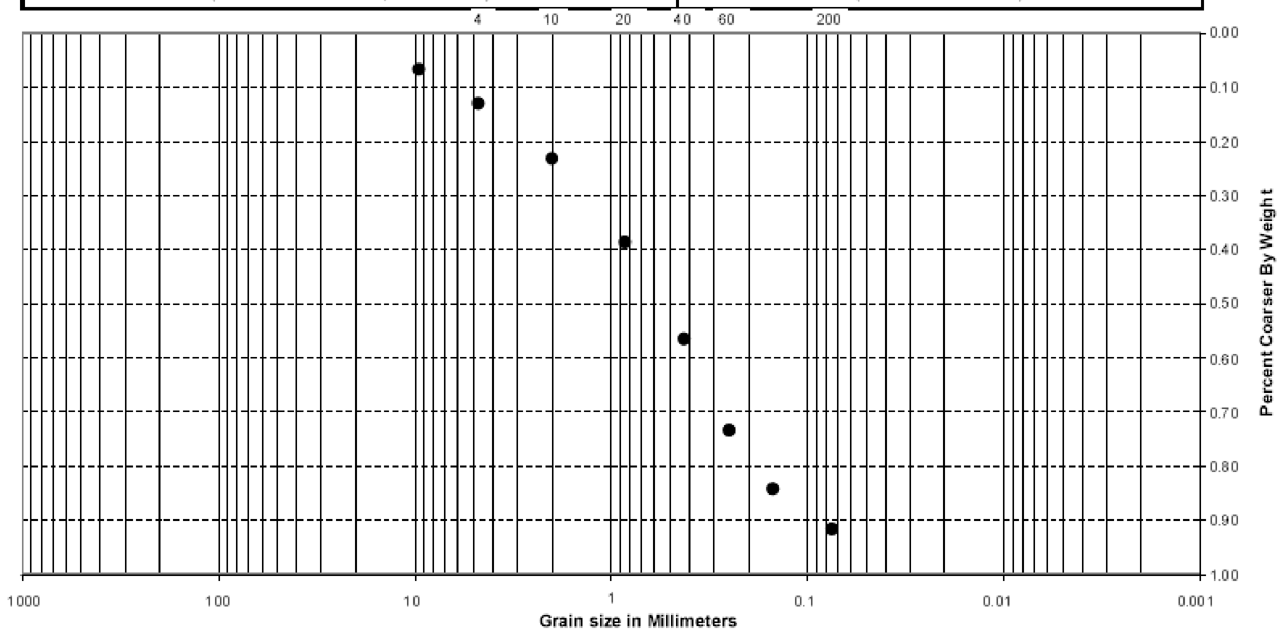
Sieve Analysis

Boring	Depth (feet)	γ_{Field} (pcf)	$\gamma_{Saturate}$ (pcf)	Fines (%)	D_{50} (mm)
B-1	50	126	138	8	0.579

Classification : Grayish brown gravelly sand (Qa)

SIEVE ANALYSIS
(Number of Mesh Per Inch, U.S. Standard)

HYDROMETER ANALYSIS
(Grain Size in Millimeters)



COBBLES	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
	GRAVEL		SAND			FINES	

Plate GD - 7

C. Y. GEOTECH, INC.

Geotechnical Engineering and Engineering Geology

Gilbert Salazar

Date : 06-2007 | P.N. No.: CYG-07-4948

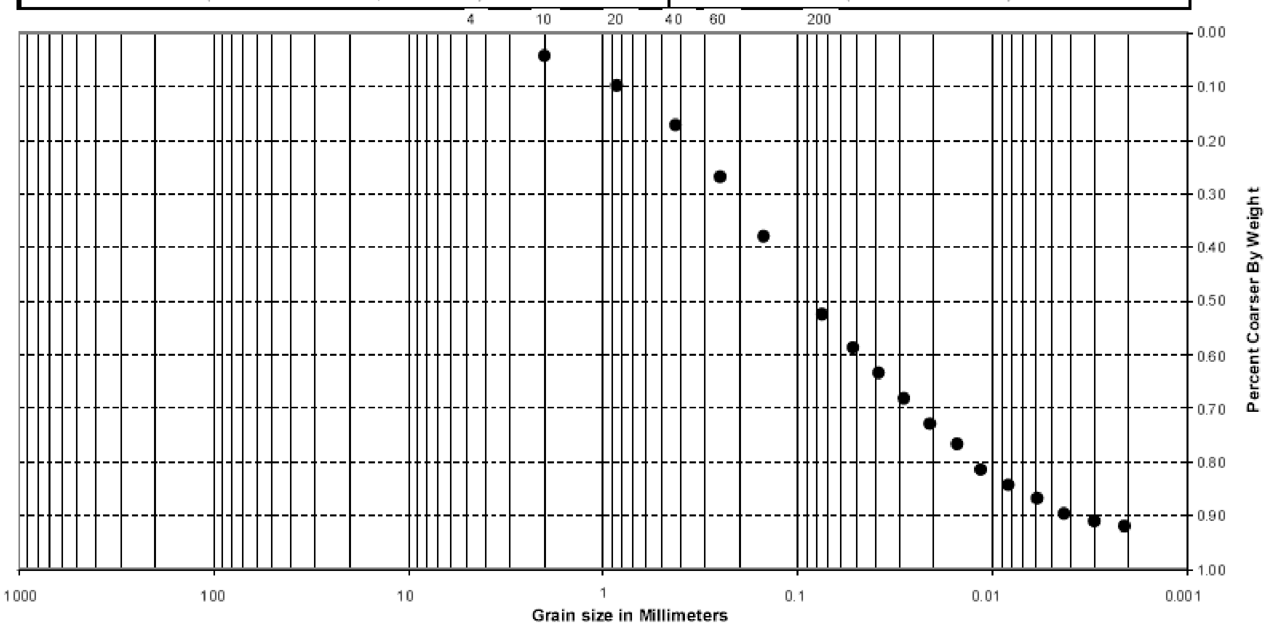
Sieve / Hydrometer Analysis

Boring	Depth (feet)	γ Field (pcf)	γ Saturate (pcf)	Fines (%)	D_{50} (mm)
B-1	60	138	138	47	0.088

Classification : Dark brown silty sand (Qa)
Clay Content = 12 %

SIEVE ANALYSIS
(Number of Mesh Per Inch, U.S. Standard)

HYDROMETER ANALYSIS
(Grain Size in Millimeters)



COBBLES	GRAVEL		SAND			FINES	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay

C. Y. GEOTECH, INC.

Geotechnical Engineering and Engineering Geology

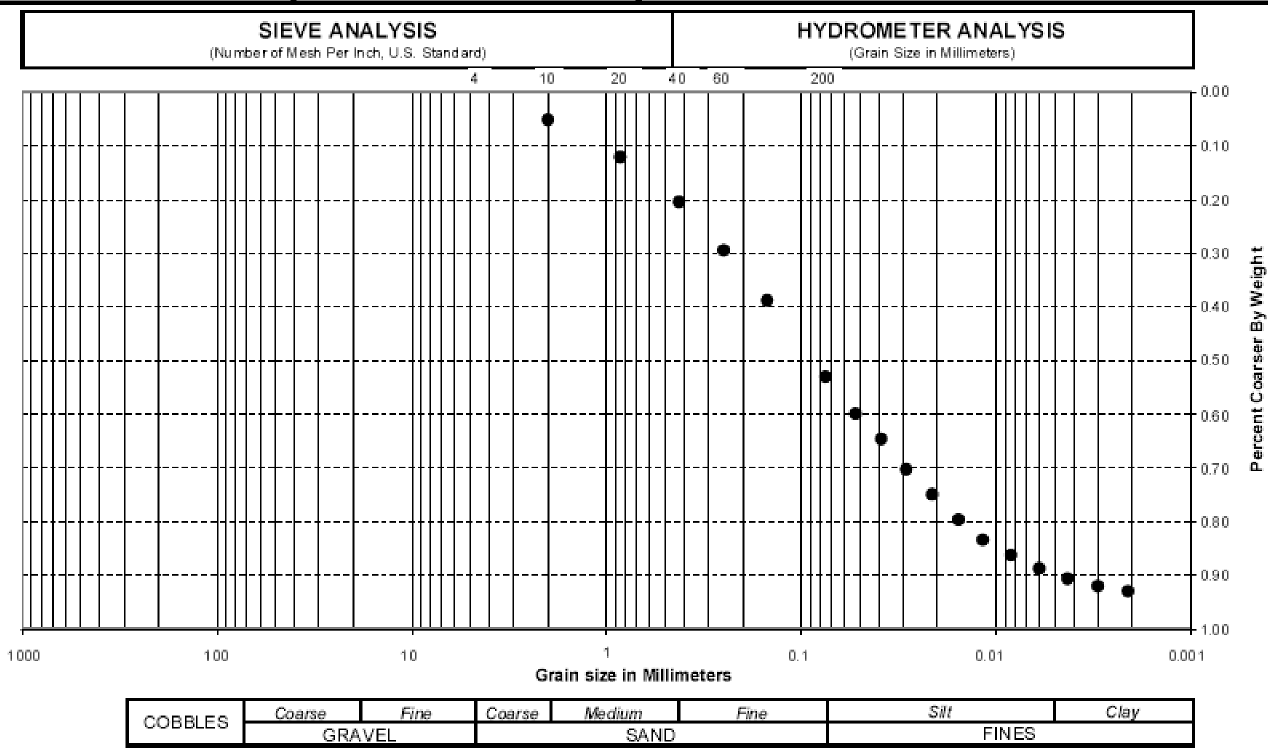
Gilbert Salazar

Date : 06-2007 | P.N. No.: CYG-07-4948

Sieve / Hydrometer Analysis

Boring	Depth (feet)	γ_{Field} (pcf)	$\gamma_{Saturate}$ (pcf)	Fines (%)	D_{50} (mm)
B-1	65	137	137	47	0.091

Classification : Brown silty sand (Qa)
Clay Content = 10 %



C. Y. GEOTECH, INC.

Geotechnical Engineering and Engineering Geology

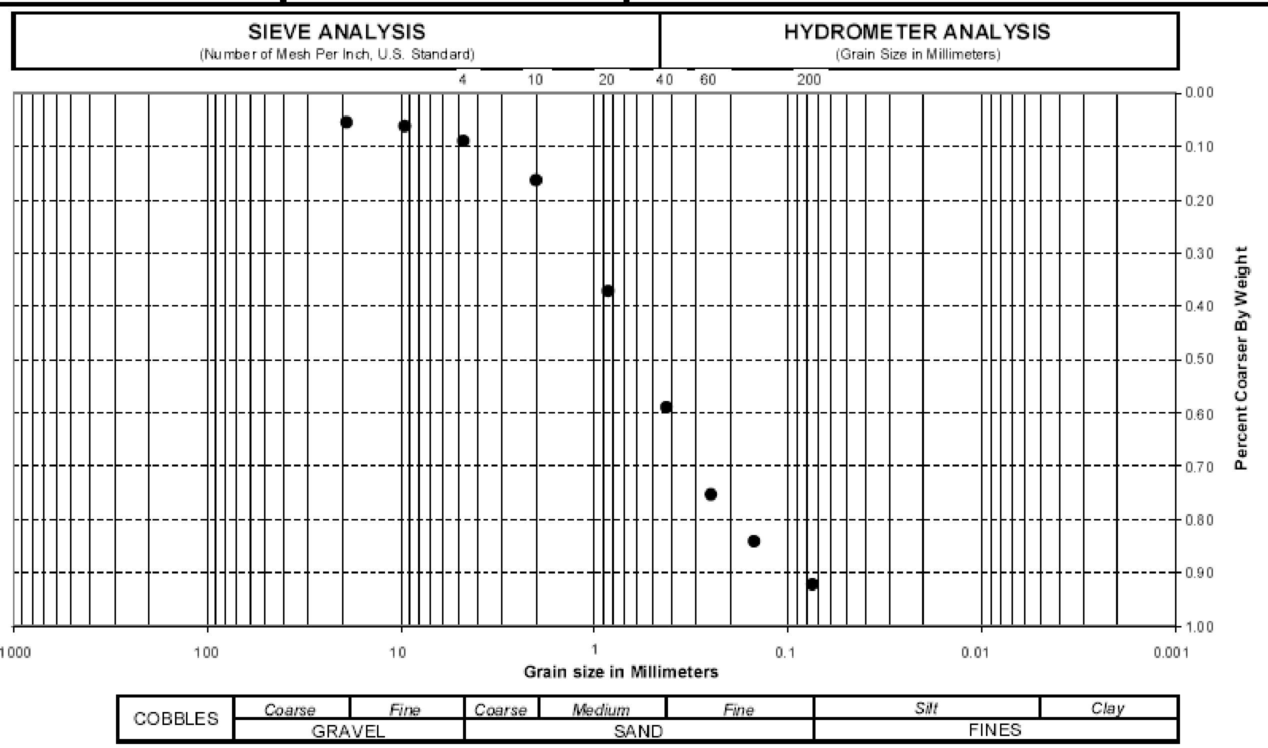
Gilbert Salazar

Date : 06-2007 | P.N. No.: CYG-07-4948

Sieve Analysis

Boring	Depth (feet)	γ_{Field} (pcf)	$\gamma_{Saturate}$ (pcf)	Fines (%)	D_{50} (mm)
B-1	70	129	140	8	0.600

Classification : Grayish brown gravelly sand (Qa)



APPENDIX B

LIQUEFACTION EVALUATION

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*                                 *
*          E Q F A U L T          *
*                                 *
*            Ver. 2.01            *
*                                 *
*                                 *
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(Estimation of Peak Horizontal Acceleration
From Digitized California Faults)

SEARCH PERFORMED FOR: GILBERT SALAZAR

JOB NUMBER: CYG-07-4948

JOB NAME: 13005-13069 VICTORY BLVD., VAN NUYS

SITE COORDINATES:
LATITUDE: 34.187 N
LONGITUDE: 118.4168 W

SEARCH RADIUS: 50 mi

ATTENUATION RELATION: 2) Campbell (1991R) Horiz. - Deep Soil & Soft Rock

UNCERTAINTY (M=Mean, S=Mean+1-Sigma): S

SCOND: 0

COMPUTE PEAK HORIZONTAL ACCELERATION

FAULT-DATA FILE USED: CALIFLT.DAT

SOURCE OF DEPTH VALUES (A=Attenuation File, F=Fault Data File): A

DETERMINISTIC SITE PARAMETERS

ABBREVIATED FAULT NAME	APPROX. DISTANCE mi (km)	MAX. CREDIBLE EVENT			MAX. PROBABLE EVENT		
		MAX. CRED. MAG.	PEAK SITE ACC. g	SITE INTENS MM	MAX. PROB. MAG.	PEAK SITE ACC. g	SITE INTENS MM
ANACAPA	26 (42)	7.00	0.158	VIII	5.00	0.042	VI
ARROYO PARIDA - MORE RANCH	44 (71)	7.50	0.108	VII	5.25	0.023	IV
CATALINA ESCARPMENT	49 (79)	7.00	0.052	VI	6.25	0.029	V
CHINO	39 (63)	7.00	0.091	VII	5.50	0.034	V
CLEARWATER	27 (44)	7.00	0.149	VIII	3.00	0.009	III
CUCAMONGA	34 (55)	7.00	0.109	VII	6.25	0.063	VI
ELYSIAN PARK SEISMIC ZONE	10 (15)	7.00	0.463	X	5.75	0.240	IX
FRAZIER MOUNTAIN	48 (77)	6.50	0.046	VI	3.00	0.004	I
GLN.HELEN-LYTLE CR-CLREMNT	46 (73)	7.00	0.058	VI	6.50	0.040	V
HOLSER	18 (28)	6.60	0.194	VIII	5.75	0.122	VII
MALIBU COAST	14 (23)	7.50	0.429	X	6.50	0.235	IX
MID-CHANNEL	49 (80)	7.50	0.091	VII	5.50	0.023	IV
NEWPORT-INGLEWOOD-OFFSHORE	9 (15)	7.00	0.383	X	5.75	0.200	VIII
NORTHRIDGE HILLS	4 (7)	6.50	0.639	X	5.00	0.244	IX
OAK RIDGE (Offshore)	42 (67)	7.20	0.095	VII	5.50	0.030	V
OAK RIDGE (Onshore)	23 (36)	7.20	0.216	VIII	6.50	0.134	VIII
PALOS VERD-CORON.B.-A.BLAN	19 (31)	7.50	0.259	IX	6.75	0.160	VIII
PINE MOUNTAIN	36 (58)	7.00	0.101	VII	4.25	0.015	IV
RAYMOND	12 (19)	7.50	0.506	X	4.00	0.056	VI
SAN ANDREAS (Mojave)	30 (49)	8.30	0.253	IX	8.00	0.210	VIII
SAN CAYETANO	26 (41)	7.50	0.227	IX	6.25	0.096	VII
SAN GABRIEL	11 (18)	7.00	0.330	IX	5.75	0.168	VIII
SANTA MONICA - HOLLYWOOD	8 (13)	7.50	0.660	XI	5.25	0.196	VIII
SANTA SUSANA	10 (15)	7.00	0.459	X	6.00	0.286	IX
SANTA YNEZ (East)	38 (62)	7.50	0.108	VII	5.25	0.023	IV

DETERMINISTIC SITE PARAMETERS

Page 2

ABBREVIATED FAULT NAME	APPROX. DISTANCE mi (km)	MAX. CREDIBLE EVENT			MAX. PROBABLE EVENT		
		MAX. CRED. MAG.	PEAK SITE ACC. g	SITE INTENS MM	MAX. PROB. MAG.	PEAK SITE ACC. g	SITE INTENS MM
SIERRA MADRE-SAN FERNANDO	7 (11)	7.50	0.712	XI	6.00	0.399	X
SIMI - SANTA ROSA	16 (26)	7.00	0.281	IX	5.25	0.094	VII
VENTURA - PITAS POINT	44 (71)	7.20	0.087	VII	5.75	0.033	V
VERDUGO	3 (5)	6.70	0.760	XI	4.50	0.195	VIII
WHITTIER - NORTH ELSINORE	13 (22)	7.50	0.362	IX	6.00	0.162	VIII

****

-END OF SEARCH- 30 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE VERDUGO FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 3.3 MILES AWAY.

LARGEST MAXIMUM-CREDIBLE SITE ACCELERATION: 0.760 g

LARGEST MAXIMUM-PROBABLE SITE ACCELERATION: 0.399 g

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*                     *
*   SOIL PROFILE LOG  *
*                     *
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SOIL PROFILE NAME: 494801

LAYER #	BASE DEPTH (ft)	SPT FIELD-N (blows/ft)	LIQUEFACTION SUSCEPTIBILITY	WET UNIT WT. (pcf)	FINES %<#200	D (mm) 50	DEPTH OF SPT (ft)
1	7.5	27.0	SUSCEPTIBLE (1)	124.0	9.0	0.270	5.25
2	12.5	28.0	SUSCEPTIBLE (1)	128.0	45.0	0.090	10.25
3	17.5	23.0	SUSCEPTIBLE (1)	130.0	9.0	0.270	15.25
4	22.5	25.0	SUSCEPTIBLE (1)	126.0	41.0	0.120	20.25
5	27.5	27.0	SUSCEPTIBLE (1)	134.0	52.0	0.150	25.25
6	32.5	8.0	UNSUSCEPTIBLE (0)	138.0	69.0	0.026	30.25
7	37.5	10.0	UNSUSCEPTIBLE (0)	137.0	69.0	0.026	35.25
8	42.5	61.0	SUSCEPTIBLE (1)	148.0	11.0	0.790	40.25
9	47.5	53.0	SUSCEPTIBLE (1)	141.0	11.0	0.790	45.25
10	52.5	44.0	SUSCEPTIBLE (1)	138.0	9.0	0.560	50.25
11	57.5	33.0	SUSCEPTIBLE (1)	138.0	48.0	0.800	55.25
12	62.5	35.0	SUSCEPTIBLE (1)	138.0	48.0	0.800	60.25
13	67.5	38.0	SUSCEPTIBLE (1)	137.0	47.0	0.560	65.25
14	72.5	59.0	SUSCEPTIBLE (1)	140.0	8.0	0.560	70.25
15	77.5	90.0	SUSCEPTIBLE (1)	136.0	8.0	0.560	75.25

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*                *
* L I Q U E F Y 2 *
*                *
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EMPIRICAL PREDICTION OF
EARTHQUAKE-INDUCED LIQUEFACTION POTENTIAL

JOB NUMBER: CYG-07-4948

JOB NAME: 13005-13069 Victory Blvd., Van Nuys, CA

LIQUEFACTION CALCULATION NAME:

SOIL-PROFILE NAME: 494801

GROUND WATER DEPTH: 20.0 ft

DESIGN EARTHQUAKE MAGNITUDE: 7.50

SITE PEAK GROUND ACCELERATION: 0.520 g

K sigma BOUND: M

rd BOUND: M

N60 CORRECTION: 1.00

FIELD SPT N-VALUES < 10 FT DEEP ARE CORRECTED FOR SHORT LENGTH OF DRIVE RODS

NOTE: Relative density values listed below are estimated using equations of Giuliani and Nicoll (1982).

LIQUEFACTION ANALYSIS SUMMARY

Seed and Others [1985] Method

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	Est.D r (%)	C N	CORR. (N1)60 (B/ft)	LIQUE. STRESS RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
1	0.25	0.016	0.016	27	95	@	@	@	@	@	@ @
1	0.75	0.047	0.047	27	95	@	@	@	@	@	@ @
1	1.25	0.078	0.078	27	95	@	@	@	@	@	@ @
1	1.75	0.109	0.109	27	95	@	@	@	@	@	@ @
1	2.25	0.140	0.140	27	95	@	@	@	@	@	@ @
1	2.75	0.171	0.171	27	95	@	@	@	@	@	@ @
1	3.25	0.202	0.202	27	95	@	@	@	@	@	@ @
1	3.75	0.233	0.233	27	95	@	@	@	@	@	@ @
1	4.25	0.264	0.264	27	95	@	@	@	@	@	@ @
1	4.75	0.295	0.295	27	95	@	@	@	@	@	@ @
1	5.25	0.326	0.326	27	95	@	@	@	@	@	@ @
1	5.75	0.357	0.357	27	95	@	@	@	@	@	@ @
1	6.25	0.388	0.388	27	95	@	@	@	@	@	@ @
1	6.75	0.419	0.419	27	95	@	@	@	@	@	@ @
1	7.25	0.450	0.450	27	95	@	@	@	@	@	@ @
2	7.75	0.481	0.481	28	86	@	@	@	@	@	@ @
2	8.25	0.513	0.513	28	86	@	@	@	@	@	@ @
2	8.75	0.545	0.545	28	86	@	@	@	@	@	@ @
2	9.25	0.577	0.577	28	86	@	@	@	@	@	@ @
2	9.75	0.609	0.609	28	86	@	@	@	@	@	@ @
2	10.25	0.641	0.641	28	86	@	@	@	@	@	@ @
2	10.75	0.673	0.673	28	86	@	@	@	@	@	@ @
2	11.25	0.705	0.705	28	86	@	@	@	@	@	@ @
2	11.75	0.737	0.737	28	86	@	@	@	@	@	@ @
2	12.25	0.769	0.769	28	86	@	@	@	@	@	@ @
3	12.75	0.801	0.801	23	72	@	@	@	@	@	@ @
3	13.25	0.834	0.834	23	72	@	@	@	@	@	@ @
3	13.75	0.866	0.866	23	72	@	@	@	@	@	@ @
3	14.25	0.899	0.899	23	72	@	@	@	@	@	@ @
3	14.75	0.931	0.931	23	72	@	@	@	@	@	@ @
3	15.25	0.964	0.964	23	72	@	@	@	@	@	@ @
3	15.75	0.996	0.996	23	72	@	@	@	@	@	@ @
3	16.25	1.029	1.029	23	72	@	@	@	@	@	@ @
3	16.75	1.061	1.061	23	72	@	@	@	@	@	@ @
3	17.25	1.094	1.094	23	72	@	@	@	@	@	@ @
4	17.75	1.126	1.126	25	70	@	@	@	@	@	@ @
4	18.25	1.157	1.157	25	70	@	@	@	@	@	@ @
4	18.75	1.189	1.189	25	70	@	@	@	@	@	@ @
4	19.25	1.220	1.220	25	70	@	@	@	@	@	@ @
4	19.75	1.252	1.252	25	70	@	@	@	@	@	@ @
4	20.25	1.283	1.275	25	70	0.904	22.6	Infin	0.957	0.325	Infin
4	20.75	1.315	1.291	25	70	0.904	22.6	Infin	0.955	0.329	Infin
4	21.25	1.346	1.307	25	70	0.904	22.6	Infin	0.954	0.332	Infin
4	21.75	1.378	1.323	25	70	0.904	22.6	Infin	0.952	0.335	Infin
4	22.25	1.409	1.339	25	70	0.904	22.6	Infin	0.951	0.338	Infin
5	22.75	1.442	1.356	27	71	0.862	23.3	Infin	0.949	0.341	Infin

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	Est.D r (%)	C N	CORR. (N1)60 (B/ft)	LIQUE. STRESS RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
5	23.25	1.475	1.374	27	71	0.862	23.3	Infin	0.947	0.344	Infin
5	23.75	1.509	1.392	27	71	0.862	23.3	Infin	0.946	0.347	Infin
5	24.25	1.542	1.410	27	71	0.862	23.3	Infin	0.944	0.349	Infin
5	24.75	1.576	1.428	27	71	0.862	23.3	Infin	0.943	0.352	Infin
5	25.25	1.609	1.445	27	71	0.862	23.3	Infin	0.941	0.354	Infin
5	25.75	1.643	1.463	27	71	0.862	23.3	Infin	0.939	0.356	Infin
5	26.25	1.676	1.481	27	71	0.862	23.3	Infin	0.937	0.358	Infin
5	26.75	1.710	1.499	27	71	0.862	23.3	Infin	0.934	0.360	Infin
5	27.25	1.743	1.517	27	71	0.862	23.3	Infin	0.932	0.362	Infin
6	27.75	1.777	1.535	8	~	~	~	~	~	~	~~
6	28.25	1.812	1.554	8	~	~	~	~	~	~	~~
6	28.75	1.846	1.573	8	~	~	~	~	~	~	~~
6	29.25	1.881	1.592	8	~	~	~	~	~	~	~~
6	29.75	1.915	1.611	8	~	~	~	~	~	~	~~
6	30.25	1.950	1.630	8	~	~	~	~	~	~	~~
6	30.75	1.984	1.649	8	~	~	~	~	~	~	~~
6	31.25	2.019	1.668	8	~	~	~	~	~	~	~~
6	31.75	2.053	1.687	8	~	~	~	~	~	~	~~
6	32.25	2.088	1.706	8	~	~	~	~	~	~	~~
7	32.75	2.122	1.724	10	~	~	~	~	~	~	~~
7	33.25	2.156	1.743	10	~	~	~	~	~	~	~~
7	33.75	2.191	1.762	10	~	~	~	~	~	~	~~
7	34.25	2.225	1.780	10	~	~	~	~	~	~	~~
7	34.75	2.259	1.799	10	~	~	~	~	~	~	~~
7	35.25	2.293	1.818	10	~	~	~	~	~	~	~~
7	35.75	2.328	1.836	10	~	~	~	~	~	~	~~
7	36.25	2.362	1.855	10	~	~	~	~	~	~	~~
7	36.75	2.396	1.874	10	~	~	~	~	~	~	~~
7	37.25	2.430	1.892	10							
8	37.75	2.466	1.912	61	97	0.740	45.1	Infin	0.870	0.379	Infin
8	38.25	2.503	1.934	61	97	0.740	45.1	Infin	0.866	0.379	Infin
8	38.75	2.540	1.955	61	97	0.740	45.1	Infin	0.862	0.379	Infin
8	39.25	2.577	1.976	61	97	0.740	45.1	Infin	0.858	0.378	Infin
8	39.75	2.614	1.998	61	97	0.740	45.1	Infin	0.855	0.378	Infin
8	40.25	2.651	2.019	61	97	0.740	45.1	Infin	0.850	0.377	Infin
8	40.75	2.688	2.041	61	97	0.740	45.1	Infin	0.845	0.376	Infin
8	41.25	2.725	2.062	61	97	0.740	45.1	Infin	0.840	0.375	Infin
8	41.75	2.762	2.083	61	97	0.740	45.1	Infin	0.836	0.374	Infin
8	42.25	2.799	2.105	61	97	0.740	45.1	Infin	0.831	0.373	Infin
9	42.75	2.835	2.125	53	88	0.707	37.5	Infin	0.826	0.372	Infin
9	43.25	2.870	2.145	53	88	0.707	37.5	Infin	0.821	0.371	Infin
9	43.75	2.906	2.165	53	88	0.707	37.5	Infin	0.816	0.370	Infin
9	44.25	2.941	2.184	53	88	0.707	37.5	Infin	0.811	0.369	Infin
9	44.75	2.976	2.204	53	88	0.707	37.5	Infin	0.806	0.368	Infin
9	45.25	3.011	2.224	53	88	0.707	37.5	Infin	0.801	0.367	Infin
9	45.75	3.047	2.243	53	88	0.707	37.5	Infin	0.796	0.366	Infin
9	46.25	3.082	2.263	53	88	0.707	37.5	Infin	0.791	0.364	Infin
9	46.75	3.117	2.283	53	88	0.707	37.5	Infin	0.786	0.363	Infin
9	47.25	3.152	2.302	53	88	0.707	37.5	Infin	0.781	0.361	Infin
10	47.75	3.187	2.321	44	78	0.680	29.9	Infin	0.776	0.360	Infin
10	48.25	3.222	2.340	44	78	0.680	29.9	Infin	0.771	0.359	Infin

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	Est.D r (%)	C N	CORR. (N1)60 (B/ft)	LIQUE. STRESS RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
10	48.75	3.256	2.359	44	78	0.680	29.9	Infin	0.765	0.357	Infin
10	49.25	3.291	2.378	44	78	0.680	29.9	Infin	0.760	0.356	Infin
10	49.75	3.325	2.397	44	78	0.680	29.9	Infin	0.755	0.354	Infin
10	50.25	3.360	2.416	44	78	0.680	29.9	Infin	0.750	0.353	Infin
10	50.75	3.394	2.435	44	78	0.680	29.9	Infin	0.745	0.351	Infin
10	51.25	3.429	2.454	44	78	0.680	29.9	Infin	0.741	0.350	Infin
10	51.75	3.463	2.473	44	78	0.680	29.9	Infin	0.736	0.348	Infin
10	52.25	3.498	2.492	44	78	0.680	29.9	Infin	0.731	0.347	Infin
11	52.75	3.532	2.510	33	66	0.654	21.6	Infin	0.726	0.345	Infin
11	53.25	3.567	2.529	33	66	0.654	21.6	Infin	0.722	0.344	Infin
11	53.75	3.601	2.548	33	66	0.654	21.6	Infin	0.717	0.342	Infin
11	54.25	3.636	2.567	33	66	0.654	21.6	Infin	0.712	0.341	Infin
11	54.75	3.670	2.586	33	66	0.654	21.6	Infin	0.707	0.339	Infin
11	55.25	3.705	2.605	33	66	0.654	21.6	Infin	0.703	0.338	Infin
11	55.75	3.739	2.624	33	66	0.654	21.6	Infin	0.698	0.336	Infin
11	56.25	3.774	2.643	33	66	0.654	21.6	Infin	0.694	0.335	Infin
11	56.75	3.808	2.662	33	66	0.654	21.6	Infin	0.689	0.333	Infin
11	57.25	3.843	2.681	33	66	0.654	21.6	Infin	0.684	0.332	Infin
12	57.75	3.877	2.699	35	66	0.629	22.0	Infin	0.680	0.330	Infin
12	58.25	3.912	2.718	35	66	0.629	22.0	Infin	0.675	0.328	Infin
12	58.75	3.946	2.737	35	66	0.629	22.0	Infin	0.671	0.327	Infin
12	59.25	3.981	2.756	35	66	0.629	22.0	Infin	0.666	0.325	Infin
12	59.75	4.015	2.775	35	66	0.629	22.0	Infin	0.661	0.323	Infin
12	60.25	4.050	2.794	35	66	0.629	22.0	Infin	0.657	0.322	Infin
12	60.75	4.084	2.813	35	66	0.629	22.0	Infin	0.654	0.321	Infin
12	61.25	4.119	2.832	35	66	0.629	22.0	Infin	0.650	0.320	Infin
12	61.75	4.153	2.851	35	66	0.629	22.0	Infin	0.646	0.318	Infin
12	62.25	4.188	2.870	35	66	0.629	22.0	Infin	0.643	0.317	Infin
13	62.75	4.222	2.888	38	68	0.609	23.1	Infin	0.639	0.316	Infin
13	63.25	4.256	2.907	38	68	0.609	23.1	Infin	0.635	0.314	Infin
13	63.75	4.291	2.926	38	68	0.609	23.1	Infin	0.632	0.313	Infin
13	64.25	4.325	2.944	38	68	0.609	23.1	Infin	0.628	0.312	Infin
13	64.75	4.359	2.963	38	68	0.609	23.1	Infin	0.624	0.311	Infin
13	65.25	4.393	2.982	38	68	0.609	23.1	Infin	0.621	0.309	Infin
13	65.75	4.428	3.000	38	68	0.609	23.1	Infin	0.618	0.308	Infin
13	66.25	4.462	3.019	38	68	0.609	23.1	Infin	0.615	0.307	Infin
13	66.75	4.496	3.038	38	68	0.609	23.1	Infin	0.612	0.306	Infin
13	67.25	4.530	3.056	38	68	0.609	23.1	Infin	0.609	0.305	Infin
14	67.75	4.565	3.075	59	83	0.591	34.8	Infin	0.606	0.304	Infin
14	68.25	4.600	3.095	59	83	0.591	34.8	Infin	0.603	0.303	Infin
14	68.75	4.635	3.114	59	83	0.591	34.8	Infin	0.600	0.302	Infin
14	69.25	4.670	3.133	59	83	0.591	34.8	Infin	0.597	0.301	Infin
14	69.75	4.705	3.153	59	83	0.591	34.8	Infin	0.595	0.300	Infin
14	70.25	4.740	3.172	59	83	0.591	34.8	Infin	0.592	0.299	Infin
14	70.75	4.775	3.192	59	83	0.591	34.8	Infin	0.589	0.298	Infin
14	71.25	4.810	3.211	59	83	0.591	34.8	Infin	0.587	0.297	Infin
14	71.75	4.845	3.230	59	83	0.591	34.8	Infin	0.584	0.296	Infin
14	72.25	4.880	3.250	59	83	0.591	34.8	Infin	0.582	0.295	Infin
15	72.75	4.915	3.269	90	100	0.574	51.6	Infin	0.580	0.295	Infin
15	73.25	4.949	3.287	90	100	0.574	51.6	Infin	0.577	0.294	Infin
15	73.75	4.983	3.306	90	100	0.574	51.6	Infin	0.575	0.293	Infin

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	Est.D r (%)	C N	CORR. (N1)60 (B/ft)	LIQUE. STRESS RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
15	74.25	5.017	3.324	90	100	0.574	51.6	Infin	0.572	0.292	Infin
15	74.75	5.051	3.342	90	100	0.574	51.6	Infin	0.570	0.291	Infin
15	75.25	5.085	3.361	90	100	0.574	51.6	Infin	0.568	0.290	Infin
15	75.75	5.119	3.379	90	100	0.574	51.6	Infin	0.566	0.290	Infin
15	76.25	5.153	3.398	90	100	0.574	51.6	Infin	0.564	0.289	Infin
15	76.75	5.187	3.416	90	100	0.574	51.6	Infin	0.562	0.288	Infin
15	77.25	5.221	3.434	90	100	0.574	51.6	Infin	0.560	0.288	Infin

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*   SOIL PROFILE LOG   *
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SOIL PROFILE NAME: 494811

LAYER #	BASE DEPTH (ft)	SPT FIELD-N (blows/ft)	LIQUEFACTION SUSCEPTIBILITY	WET UNIT WT. (pcf)	FINES %<#200	D (mm) 50	DEPTH OF SPT (ft)
1	7.5	10.0	UNSUSCEPTIBLE (0)	137.0	69.0	0.026	5.25
2	12.5	61.0	SUSCEPTIBLE (1)	148.0	11.0	0.790	12.25
3	17.5	53.0	SUSCEPTIBLE (1)	141.0	11.0	0.790	15.25
4	22.5	44.0	SUSCEPTIBLE (1)	138.0	9.0	0.560	20.25
5	27.5	33.0	SUSCEPTIBLE (1)	138.0	48.0	0.800	25.25
6	32.5	35.0	SUSCEPTIBLE (1)	138.0	48.0	0.800	30.25
7	37.5	38.0	SUSCEPTIBLE (1)	137.0	47.0	0.560	35.25
8	42.5	59.0	SUSCEPTIBLE (1)	140.0	8.0	0.560	45.25
9	47.5	90.0	SUSCEPTIBLE (1)	136.0	8.0	0.560	45.25
10	52.5	56.0	SUSCEPTIBLE (1)	136.0	8.0	0.560	50.25

Overburden was calculated from the bottom of basement
Groundwater surface was assumed at the bottom of basement

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* L I Q U E F Y 2 *
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EMPIRICAL PREDICTION OF
EARTHQUAKE-INDUCED LIQUEFACTION POTENTIAL

JOB NUMBER: CYG-07-4948

JOB NAME: 13005-13069 Victory Blvd., Van Nuys, CA

LIQUEFACTION CALCULATION NAME: Galbert Salazar

SOIL-PROFILE NAME: 494811

GROUND WATER DEPTH: 0.0 ft

DESIGN EARTHQUAKE MAGNITUDE: 7.50

SITE PEAK GROUND ACCELERATION: 0.520 g

K sigma BOUND: M

rd BOUND: M

N60 CORRECTION: 1.00

FIELD SPT N-VALUES < 10 FT DEEP ARE CORRECTED FOR SHORT LENGTH OF DRIVE RODS

NOTE: Relative density values listed below are estimated using equations of Giuliani and Nicoll (1982).

Overburden was calculated from the bottom of basement
Groundwater surface was assumed at the bottom of basement

LIQUEFACTION ANALYSIS SUMMARY

Seed and Others [1985] Method

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	Est. D_r (%)	C_N	CORR. (N1)60 (B/ft)	LIQUE. STRESS RATIO	r_d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
1	0.25	0.017	0.009	10	~	~	~	~	~	~	~~
1	0.75	0.051	0.028	10	~	~	~	~	~	~	~~
1	1.25	0.086	0.047	10	~	~	~	~	~	~	~~
1	1.75	0.120	0.065	10	~	~	~	~	~	~	~~
1	2.25	0.154	0.084	10	~	~	~	~	~	~	~~
1	2.75	0.188	0.103	10	~	~	~	~	~	~	~~
1	3.25	0.223	0.121	10	~	~	~	~	~	~	~~
1	3.75	0.257	0.140	10	~	~	~	~	~	~	~~
1	4.25	0.291	0.159	10	~	~	~	~	~	~	~~
1	4.75	0.325	0.177	10	~	~	~	~	~	~	~~
1	5.25	0.360	0.196	10	~	~	~	~	~	~	~~
1	5.75	0.394	0.215	10	~	~	~	~	~	~	~~
1	6.25	0.428	0.233	10	~	~	~	~	~	~	~~
1	6.75	0.462	0.252	10	~	~	~	~	~	~	~~
1	7.25	0.497	0.270	10	~	~	~	~	~	~	~~
2	7.75	0.532	0.290	61	134	1.396	85.2	Infin	0.984	0.610	Infin
2	8.25	0.569	0.312	61	134	1.396	85.2	Infin	0.983	0.607	Infin
2	8.75	0.606	0.333	61	134	1.396	85.2	Infin	0.982	0.604	Infin
2	9.25	0.643	0.355	61	134	1.396	85.2	Infin	0.981	0.601	Infin
2	9.75	0.680	0.376	61	134	1.396	85.2	Infin	0.980	0.599	Infin
2	10.25	0.717	0.397	61	134	1.396	85.2	Infin	0.979	0.597	Infin
2	10.75	0.754	0.419	61	134	1.396	85.2	Infin	0.978	0.595	Infin
2	11.25	0.791	0.440	61	134	1.396	85.2	Infin	0.977	0.593	Infin
2	11.75	0.828	0.462	61	134	1.396	85.2	Infin	0.976	0.592	Infin
2	12.25	0.865	0.483	61	134	1.396	85.2	Infin	0.975	0.590	Infin
3	12.75	0.901	0.504	53	120	1.283	68.0	Infin	0.974	0.589	Infin
3	13.25	0.937	0.523	53	120	1.283	68.0	Infin	0.972	0.588	Infin
3	13.75	0.972	0.543	53	120	1.283	68.0	Infin	0.971	0.588	Infin
3	14.25	1.007	0.563	53	120	1.283	68.0	Infin	0.970	0.587	Infin
3	14.75	1.042	0.582	53	120	1.283	68.0	Infin	0.969	0.587	Infin
3	15.25	1.078	0.602	53	120	1.283	68.0	Infin	0.968	0.586	Infin
3	15.75	1.113	0.622	53	120	1.283	68.0	Infin	0.967	0.585	Infin
3	16.25	1.148	0.641	53	120	1.283	68.0	Infin	0.966	0.585	Infin
3	16.75	1.183	0.661	53	120	1.283	68.0	Infin	0.965	0.584	Infin
3	17.25	1.219	0.680	53	120	1.283	68.0	Infin	0.964	0.583	Infin
4	17.75	1.254	0.700	44	104	1.125	49.5	Infin	0.963	0.583	Infin
4	18.25	1.288	0.719	44	104	1.125	49.5	Infin	0.961	0.582	Infin
4	18.75	1.323	0.738	44	104	1.125	49.5	Infin	0.960	0.582	Infin
4	19.25	1.357	0.756	44	104	1.125	49.5	Infin	0.959	0.582	Infin
4	19.75	1.392	0.775	44	104	1.125	49.5	Infin	0.958	0.581	Infin
4	20.25	1.426	0.794	44	104	1.125	49.5	Infin	0.957	0.581	Infin
4	20.75	1.461	0.813	44	104	1.125	49.5	Infin	0.955	0.580	Infin
4	21.25	1.495	0.832	44	104	1.125	49.5	Infin	0.954	0.579	Infin
4	21.75	1.530	0.851	44	104	1.125	49.5	Infin	0.952	0.578	Infin
4	22.25	1.564	0.870	44	104	1.125	49.5	Infin	0.951	0.578	Infin
5	22.75	1.599	0.889	33	86	1.010	33.3	Infin	0.949	0.577	Infin

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	Est.D r (%)	C N	CORR. (N1)60 (B/ft)	LIQUE. STRESS RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
5	23.25	1.633	0.908	33	86	1.010	33.3	Infin	0.947	0.576	Infin
5	23.75	1.668	0.927	33	86	1.010	33.3	Infin	0.946	0.575	Infin
5	24.25	1.702	0.945	33	86	1.010	33.3	Infin	0.944	0.575	Infin
5	24.75	1.737	0.964	33	86	1.010	33.3	Infin	0.943	0.574	Infin
5	25.25	1.771	0.983	33	86	1.010	33.3	Infin	0.941	0.573	Infin
5	25.75	1.806	1.002	33	86	1.010	33.3	Infin	0.939	0.572	Infin
5	26.25	1.840	1.021	33	86	1.010	33.3	Infin	0.937	0.570	Infin
5	26.75	1.875	1.040	33	86	1.010	33.3	Infin	0.934	0.569	Infin
5	27.25	1.909	1.059	33	86	1.010	33.3	Infin	0.932	0.568	Infin
6	27.75	1.944	1.078	35	85	0.938	32.8	Infin	0.930	0.567	Infin
6	28.25	1.978	1.097	35	85	0.938	32.8	Infin	0.928	0.566	Infin
6	28.75	2.013	1.116	35	85	0.938	32.8	Infin	0.926	0.564	Infin
6	29.25	2.047	1.134	35	85	0.938	32.8	Infin	0.923	0.563	Infin
6	29.75	2.082	1.153	35	85	0.938	32.8	Infin	0.921	0.562	Infin
6	30.25	2.116	1.172	35	85	0.938	32.8	Infin	0.919	0.561	Infin
6	30.75	2.151	1.191	35	85	0.938	32.8	Infin	0.916	0.559	Infin
6	31.25	2.185	1.210	35	85	0.938	32.8	Infin	0.913	0.557	Infin
6	31.75	2.220	1.229	35	85	0.938	32.8	Infin	0.910	0.556	Infin
6	32.25	2.254	1.248	35	85	0.938	32.8	Infin	0.907	0.554	Infin
7	32.75	2.288	1.267	38	85	0.883	33.6	Infin	0.904	0.552	Infin
7	33.25	2.323	1.285	38	85	0.883	33.6	Infin	0.902	0.551	Infin
7	33.75	2.357	1.304	38	85	0.883	33.6	Infin	0.899	0.549	Infin
7	34.25	2.391	1.323	38	85	0.883	33.6	Infin	0.896	0.547	Infin
7	34.75	2.425	1.341	38	85	0.883	33.6	Infin	0.893	0.546	Infin
7	35.25	2.460	1.360	38	85	0.883	33.6	Infin	0.890	0.544	Infin
7	35.75	2.494	1.379	38	85	0.883	33.6	Infin	0.886	0.542	Infin
7	36.25	2.528	1.397	38	85	0.883	33.6	Infin	0.882	0.539	Infin
7	36.75	2.562	1.416	38	85	0.883	33.6	Infin	0.878	0.537	Infin
7	37.25	2.597	1.434	38	85	0.883	33.6	Infin	0.874	0.535	Infin
8	37.75	2.631	1.453	59	99	0.794	46.8	Infin	0.870	0.532	Infin
8	38.25	2.666	1.473	59	99	0.794	46.8	Infin	0.866	0.530	Infin
8	38.75	2.701	1.492	59	99	0.794	46.8	Infin	0.862	0.528	Infin
8	39.25	2.736	1.512	59	99	0.794	46.8	Infin	0.858	0.525	Infin
8	39.75	2.771	1.531	59	99	0.794	46.8	Infin	0.855	0.523	Infin
8	40.25	2.806	1.550	59	99	0.794	46.8	Infin	0.850	0.520	Infin
8	40.75	2.841	1.570	59	99	0.794	46.8	Infin	0.845	0.517	Infin
8	41.25	2.876	1.589	59	99	0.794	46.8	Infin	0.840	0.514	Infin
8	41.75	2.911	1.609	59	99	0.794	46.8	Infin	0.836	0.511	Infin
8	42.25	2.946	1.628	59	99	0.794	46.8	Infin	0.831	0.508	Infin
9	42.75	2.981	1.647	90	123	0.794	71.4	Infin	0.826	0.505	Infin
9	43.25	3.015	1.665	90	123	0.794	71.4	Infin	0.821	0.502	Infin
9	43.75	3.049	1.684	90	123	0.794	71.4	Infin	0.816	0.500	Infin
9	44.25	3.083	1.702	90	123	0.794	71.4	Infin	0.811	0.497	Infin
9	44.75	3.117	1.721	90	123	0.794	71.4	Infin	0.806	0.494	Infin
9	45.25	3.151	1.739	90	123	0.794	71.4	Infin	0.801	0.491	Infin
9	45.75	3.185	1.757	90	123	0.794	71.4	Infin	0.796	0.488	Infin
9	46.25	3.219	1.776	90	123	0.794	71.4	Infin	0.791	0.485	Infin
9	46.75	3.253	1.794	90	123	0.794	71.4	Infin	0.786	0.482	Infin
9	47.25	3.287	1.813	90	123	0.794	71.4	Infin	0.781	0.479	Infin
10	47.75	3.321	1.831	56	94	0.758	42.4	Infin	0.776	0.476	Infin
10	48.25	3.355	1.849	56	94	0.758	42.4	Infin	0.771	0.472	Infin

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	Est.D r (%)	C N	CORR. (N1)60 (B/ft)	LIQUE. STRESS RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
10	48.75	3.389	1.868	56	94	0.758	42.4	Infin	0.765	0.469	Infin
10	49.25	3.423	1.886	56	94	0.758	42.4	Infin	0.760	0.466	Infin
10	49.75	3.457	1.905	56	94	0.758	42.4	Infin	0.755	0.463	Infin
10	50.25	3.491	1.923	56	94	0.758	42.4	Infin	0.750	0.460	Infin
10	50.75	3.525	1.941	56	94	0.758	42.4	Infin	0.745	0.457	Infin
10	51.25	3.559	1.960	56	94	0.758	42.4	Infin	0.741	0.455	Infin
10	51.75	3.593	1.978	56	94	0.758	42.4	Infin	0.736	0.452	Infin
10	52.25	3.627	1.997	56	94	0.758	42.4	Infin	0.731	0.449	Infin