Appendix D Geotechnical Engineering Investigation

Appendix

This page intentionally left blank.

C. Y. GEOTECH, INC.

Engineering Geology and Geotechnical Engineering

9428 Eton Avenue, Unit M, Chatsworth, CA 91311
Tel: (818) 341-1899 Fax: (818) 341-1897 Email: cygeotech@sbcglobal.net

GEOTECHNICAL ENGINEERING INVESTIGATION

PERCOLATION TEST FOR WATER QUALITY CONTROL PLAN
PROPOSED AUTOMOBILE OFFICE BUILDING
AND RECREATION VEHICLE PARKING LOT
APN 027-614-430 AND 027-614-431
NORTHEAST CORNER OF SANTO ANTONIO DRIVE
AND MT. VERNON AVENUE, COLTON, CALIFORNIA

FOR

GIANT INLAND EMPIRE RV CENTER INC.

C/O MR. JOHN BRALY

INSTRUTURES DESIGN AND BUILD INC.

JANUARY 31, 2020
PROJECT NO. CYG-19-8863

January 31, 2020

P.N. CYG-19-8863

Giant Inland Empire RV Center Inc.
c/o Mr. John Braly
Instructures Design and Build Inc.
1880 century Park E. Suite 315
Los Angeles, CA 90067

Subject: Geotechnical Engineering Investigation, Percolation Test for Water Qaulity Control Plan, Proposed Automobile Office Building and Recreation Vehicle Parking Lot, APN 027-614-430 AND 027-614-431, Northeast Corner of Santo Antonio Drive and Mt. Vernon Avenue, Colton, California

Dear Mr. Braly,

Per your request, C. Y. Geotech (CYG), Inc. has performed a geotechnical engineering investigation and percolation test 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 automobile office building and recreation vehicle parking lot. The accompanying geotechnical engineering report presents the findings and conclusions of this investigation and the recommendations for the design and construction of the proposed automobile office building and recreation vehicle parking lot.

Based on the findings of this investigation, the development of the proposed automobile office building and recreation vehicle parking lot on the site is feasible from geotechnical engineering viewpoints provided the recommendations in the accompanying report are properly incorporated into design and implemented during construction. Conventional spread footing founded into compacted fill can be used to support the proposed automobile building. The asphalt concrete pavement for and recreation vehicle parking lot should be supported by a minimum of two feet of compacted fill. An infiltration rate of 5.4 inches per hour can be used in the design of infiltration system.

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

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

Gary Guo
Staff Engineer

Paul Cai
RCE 80352

REGISTERED PROFESSIONAL ENGINEER
LUO B. CAI
C 80352
EXP 12-31-20
CIVIL
STATE OF CALIFORNIA

John Tsao
RCE 46886/ CEG 1783

REGISTERED PROFESSIONAL ENGINEER
TZE-TZONG TSAO
No. C 046886
Exp. 6-30-21
CIVIL
STATE OF CALIFORNIA

ncl: Appendix A, Field Exploration and Laboratory Testing
Appendix B, Liquefaction Evaluation
Appendix C, Infiltration Test Data and Calculation
Appendix D, Settlement Evaluations

cc: (5) Addressee

GEOTECHNICAL ENGINEERING INVESTIGATION

Percolation Test for Water Qaulity Control Plan
Proposed Automobile Office Building and Recreation Vehicle Parking Lot
APN 027-614-430 AND 027-614-431, Northeast Corner of
Santo Antonio Drive and Mt. Vernon Avenue, Colton, California

As requested, CYG has performed a geotechnical engineering investigation for the subject project. The purposes of this investigation are to determine the infiltration rate of onsite soil, to evaluate the soils engineering conditions of onsite earth materials which may affect the proposed development, and to provide recommendations for the design and construction of the proposed automobile office building and recreation parking lot.

1.0 SCOPE OF WORK

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

a. Researched and reviewed available geologic and soils data of the site and its vicinity. A site location map is shown on Figure 1.

b. Logged and sampled two (2) deep borings to a maximum depth of 51 feet at the locations as shown on Plate 1 for liquefaction evaluations and foundation evaluations.

c. Logged, logged and sampled ten (10) shallow borings to a maximum depth of 12 feet at the locations as shown on Plate 1 for foundation evaluation and infiltration tests.

d. Performed infiltration tests to evaluate the infiltration rate of onsite soils for design of infiltration system.

e. Performed 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 7.0.

f. Performed faulting study and seismic evaluation. The building code seismic factors for structural design were determined.

f. Performed geotechnical engineering evaluation and analyses. The potential of earthquake-induced geologic hazards which may affect the stability of the site and the proposed development were evaluated.

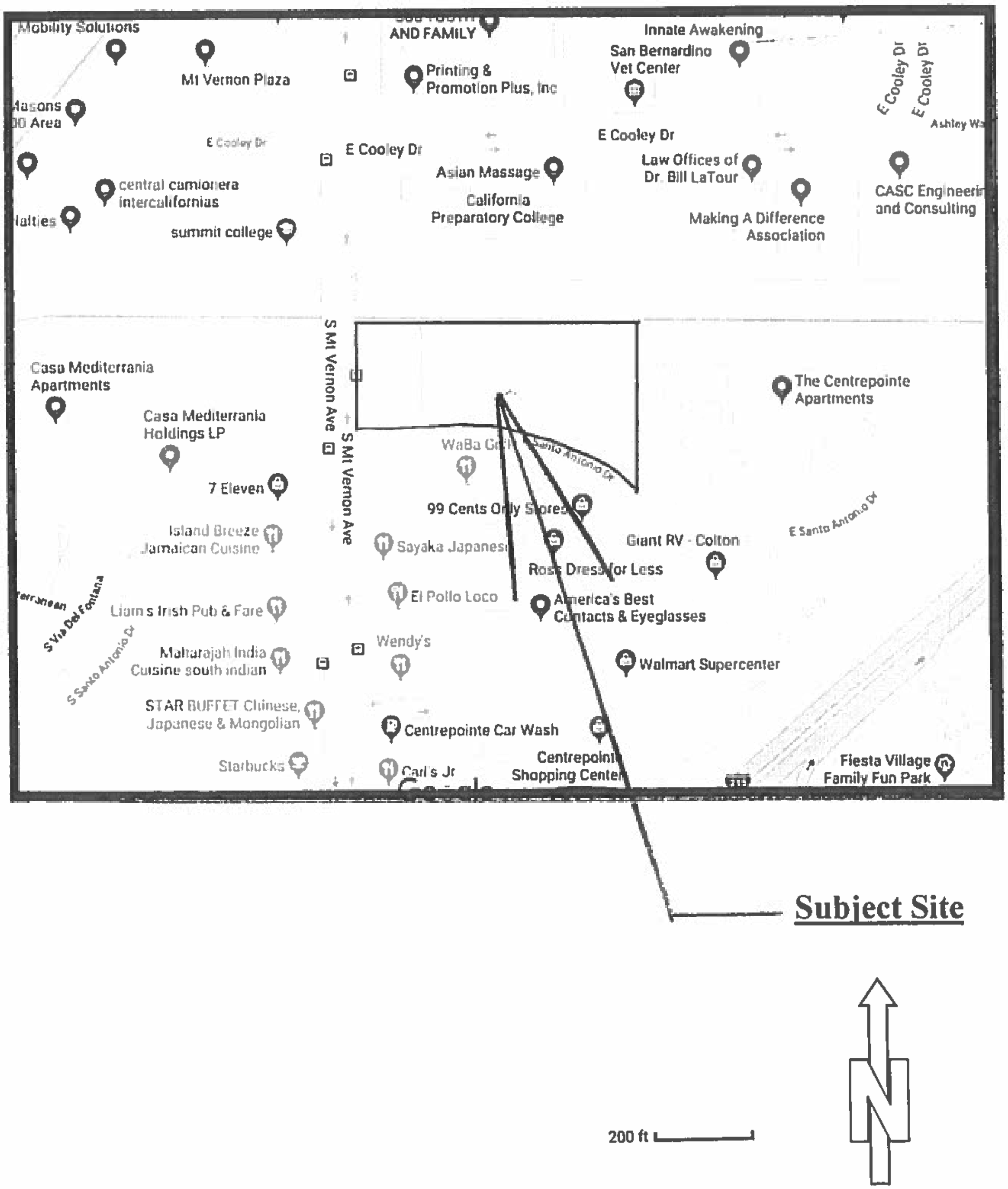
g. Prepared this soils engineering report to present the findings and conclusions of this investigation, to provide the infiltration rate of onsite soils and to provide recommendations for the design and construction of the proposed automobile building and recreation vehicle parking lot.

2.0 SITE DESCRIPTION

The site is located at the north east corner of Santo Antonio Drive and Mt. Vernon Avenue, Colton, California. The APN numbers of the site are 027-614-430 AND 027-614-431. A site location map adopted from the Google Map is shown on Figure 1. The site is bounded on the south by Santo Antonio Drive, on the west by Mt. Vernon Avenue, on the east by residential community and on the north by a flow channel. The site is fairly level, roughly trapezoidal-shaped and currently vacant.

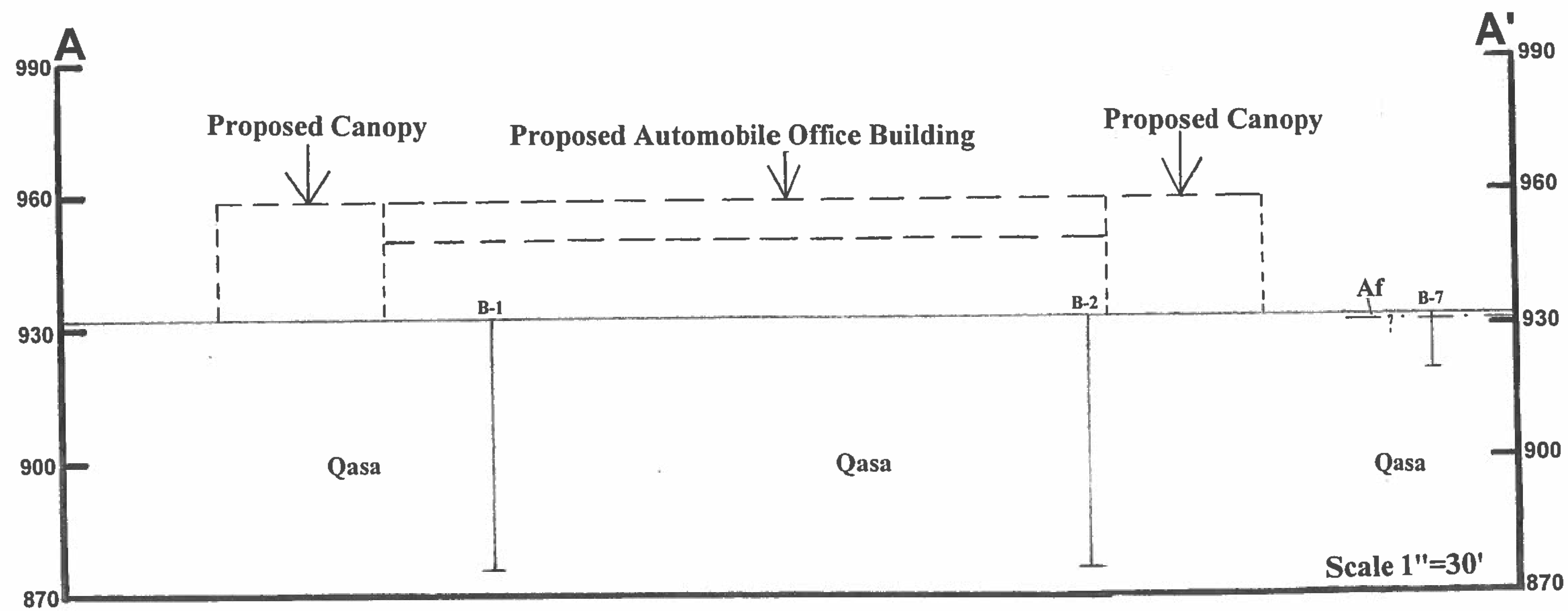
3.0 PROPOSED DEVELOPMENT

Information regarding the proposed development was provided by you and used as a guide for field exploration and report preparation. Based on the information provided, it is our understanding that the site

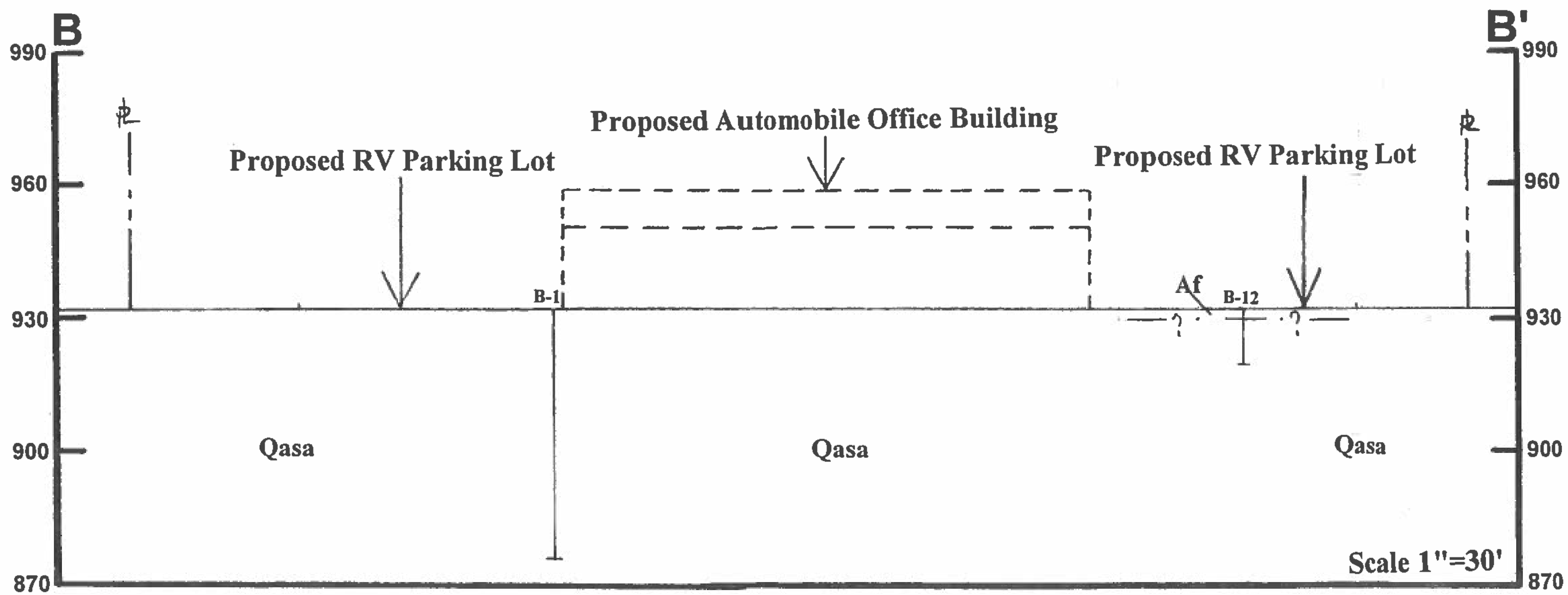


(Adopted from Google Maps)

C. Y. GEOTECH, INC. Engineering Geology and Geotechnical Engineering	Site Location Map	
	CYG-20-8863	Figure 1



C. Y. GEOTECH, INC. Engineering Geology and Geotechnical Engineering	Geotechnical Cross Section A-A'
	CYG-19-8863 | Figure 2



C. Y. GEOTECH, INC. Engineering Geology and Geotechnical Engineering	**Geotechnical Cross Section B-B'**	
	CYG-19-8863	**Figure 3**

is to be developed as a automobile office building with surrounding recreation vehicle parking lots. The automobile building is to be at-grade.

A site plan showing the site, property lines, proposed automobile office building and recreation vehicle parking lots is shown on Plate 1. Two geotechnical cross sections (A-A' and B-B') showing the site, property lines, proposed automobile office building and recreation vehicle parking lot and subsurface earth materials are shown on Figures 2 and 3.

Formal grading, architectural and structural plans for the development of the proposed automobile office building with recreation vehicle parking lot have not been prepared and await for the findings, conclusions and recommendations of this investigation. No significant grading is anticipated.

4.0 DATA RESEARCH

City of Colton and County of San Bernandino were contacted to perform data research of grading records, geologic reports and soils engineering reports for past investigation, past grading or past development the site. However, no specific geologic report or soils engineering report for past investigation, past grading or past development of the site was found from the city files and county files during our data research.

5.0 FIELD EXPLORATION AND LABORATORY TESTING

Field exploration was performed by one of our engineers and one of our geologists on January 9, 2020 and January 13, 2020 with the aid of a hollow-ste drill rig and hand laborers. Two (2) deep borings were explored to a maximum depth of 51 feet at the locations as shown on Plate 1 for liquefaction evaluations and foundation evaluations. Ten (10) shallow borings were explored to a maximum depth of 12 feet at the locations as shown on Plate 1 for foundation evaluations. Five (5) shallow borings were used for infiltration tests. The borings were logged by the engineer and geologist, and backfilled when the field exploration and percolation tests were completed. The boring logs are presented in Appendix A.

The soils encountered in the borings were sampled by using a split-tube soil sampler and a SPT soil sampler. The boring logs are presented in Plates A-1 to A-5. 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 soils were logged and then retained in plastic bags for laboratory particle size tests. The ring samples of onsite soils were retained in a series of brass rings, each having an inner diameter of 2.4 inches and a height of 1.0 inch. The soil samples and brass rings were then retained in plastic, close-fitting, moisture-tight containers. Bulk samples of onsite earth materials were collected for laboratory compaction test and expansion index test.

Laboratory testing was performed after review of the 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 laboratory testing. The following engineering properties of onsite earth materials were determined by CYG: 1) field density and field moisture content, 2) maximum dry density and optimum moisture content, 3) cohesion and friction angle, 4) compressibility and hydro-consolidation, 5) expansion index, and 6) grain size distribution. The results of laboratory tests are presented in Appendix A and summarized in Section 7.0. R value test and chemical tests of onsite soil were performed by outside laboratory testing agencies.

6.0 EARTH MATERIAL

Earth materials encountered in the borings consisted primarily of minor fill soil and alluvium. Descriptions of the fill soil and alluvial soil encountered are shown on the boring logs enclosed in Appendix A. The engineering properties of the fill soil and alluvial soil determined from laboratory tests are presented in Appendix A and summarized in Section 7.0. No bedrock was encountered in the borings. Bedrock underlies the site at depth and will not significantly affect the stability of the site and the proposed development. A regional geologic map showing the site and site vicinity adopted from the Generalized Geologic Map of Southwestern San Bernardino County is shown on Figure 4.

6.1 Artificial Fill (Af)

Artificial fill was encountered from ground surface in most of the borings and to a maximum depth of 2 feet. The fill soil consisted primarily of light brown silty sand in a dry to slightly moist and loose to moderately dense condition. The fill soil, in its present condition, is not suitable for foundation or slab support.

6.2 Alluvium (Qasa)

River channel alluvium was encountered either from ground surface or underlying fill soil in the borings. The alluvial soil consisted primarily of light brown to grayish brown silty sand and gray to grayish brown gravelly sand. Scatter layers of clayey sandy silt and sandy clayey silt were also encountered. Laboratory tests indicated that the dry density of alluvial soil varied from 92 to 136 pounds per cubic foot (pcf). The expansion index test indicated an expansion index of 0 for the tested sample of sandy alluvial soil Soils with an expansion index of zero (0) are considered as non-expansive soils.

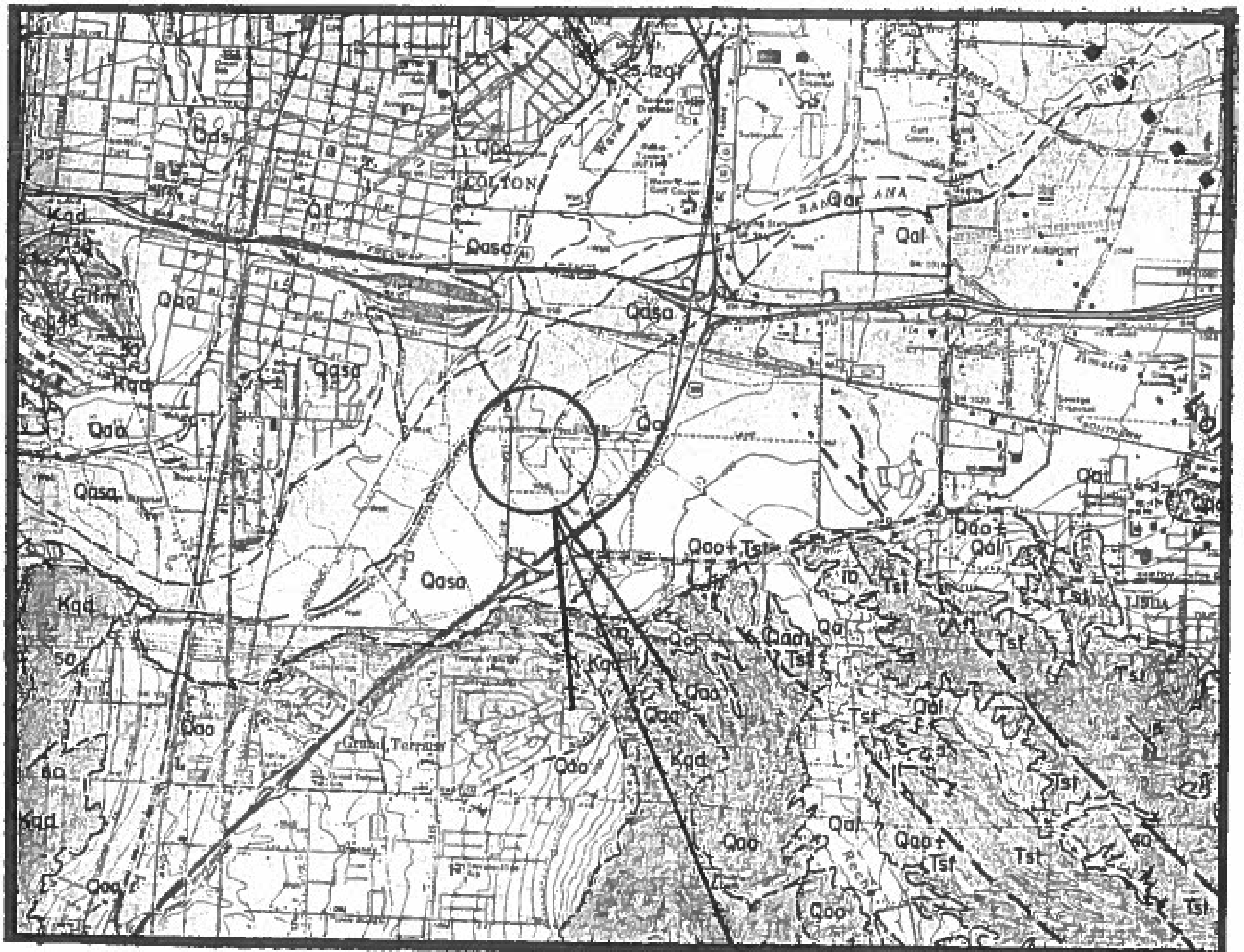
7.0 ENGINEERING PROPERTY

The engineering properties of alluvium determined from laboratory tests are presented in Appendix A and summarized below:

Field Dry Density (Qasa):	92 - 136 pcf
Field Moisture Content (Qasa):	1 - 20 %
Maximum Dry Density (Qasa):	133.5 pcf
Optimum Moisture Content (Qasa):	7.5 %
Cohesion (Qasa, peak):	60 - 120 psf
Cohesion (Qasa, residual):	20 - 50 psf
Friction Angle (Qasa, peak):	30 deg
Friction Angle (Qasa, residual):	25.5 - 28.5
Compressibility (Qasa):	See Plates CS-1 to CS-15
Hydroconsolidation (Qasa, @ 2000 psf overburden):	0 - 0.9%
Swelling (Qasa, @ 2000 psf overburden):	0 %
Expansion Index (Qasa):	EI = 0

8.0 CHEMICAL TEST

Chemical tests were performed to determine the PH value, electrical resistivity, sulphate concentration and chloride concentration of onsite soils. The samples for chemical tests were retrieved by CYG. The chemical tests were performed by Project X Corrosion Engineering. The standards of CTM 643 were used in the determination of PH value and electrical resistivity. The standards of CTM 422 were used in the determination of chloride concentration. The standards of CTM 417 were used in the determination of sulfate concentration. The electrical resistivity test was performed under a saturated condition. The results of the chemical tests are presented in Appendix A and summarized in the following table.



Subject Site

Legend

Qasa	**SANTA ANA RIVER CHANNEL ALLUVIUM** *Unconsolidated sandy alluvium of the Santa Anna River channel. Apparetly not as recently subject to fluvial reworking as the adjacent Qar.*
Qal	**YOUNGER ALLUVIUM UNDIFFERENTIATED** *Unconsolidated alluvium of the valley area and along some major drainage courses within the highlands surrounding the valley.*

N

Scale 1 = 48000

(Adopted from Generalized Geologic Map of Southwestern San Bernardino County, California)

C. Y. GEOTECH, INC. Engineering Geology and Geotechnical Engineering	**Regional Geologic Map**	
	CYG-19-8863	**Figure 4**

Sample	Sample Description	PH Value	Chloride ppm	Sulphate ppm	Resistivity ohm-cm
Older Alluvium	Brown gravelly silty sand with cobble	8.2	10.8	24.4	201,000

Certain sulfate minerals present in the soil, rockmass and groundwater have a detrimental effect on concrete. When soluble sulfate concentrations are greater than 1000 ppm in soil and 150 ppm in water, mitigation measures must be taken to protect any concrete structures in contact with the soils. The tested soil sample has a sulfate concentration of 24.4 ppm which is much less than 1000 ppm. Therefore, the tested sample has a low potential corrosion effect on concrete works caused by the chloride of onsite soil.

Large concentrations of chlorides will adversely affect ferrous materials, such as, iron and steel. When chloride concentrations exceed 18000 ppm, mitigation measures must be taken to protect any steel reinforcing within concrete and any steel pipe or cast iron that serve the development. The tested sample has a chloride concentration of 10.8 ppm which is much less than the critical value of chloride concentration. Therefore, the tested sample has a low potential corrosion effect on ferrous materials caused by the chloride of onsite soil.

Mitigation measures must be taken to protect concrete and steel in soil when the PH value gets down around 4. The chemical test indicated a PH value of 8.2 for the tested sample. Earth materials with a PH value of 8.2 will not cause adverse corrosion effect on concrete and iron.

Electrical resistivity of soil is the most common factor in determining soil corrosivity. As a soil's resistivity decreases, its corrosivity increases. Mitigation measures must be taken when test results indicate the soil to be moderately corrosive or worse per the following table.

Soil Resistivity, ohm-cm	0 - 1000	1000 - 2000	2000 - 10000	over 10000
Corrosivity Category	severely corrosive	corrosive	moderately corrosive	mildly corrosive

The chemical test indicated that the tested sample has an electrical resistivity of 201,000 ohm-cm which is much greater than 10,000 ohm-cm and therefore, is considered mildly corrosive.

Based on the results of the chemical test, it is the opinion of CYG that onsite soil is considered as mildly corrosive and will not have significant adverse corrosion effect on concrete works and underground utility metal pipes. Regular cement such as Portland Cement Type II can be used for concrete works.

9.0 SURFACE AND SUBSURFACE WATER

No surface water was observed on the site during field exploration. Surface water is limited to the precipitation falling directly on the site. Seeps, seepages and groundwater were not encountered in any of the borings to the depth explored. In accordance with the water data library of the California Department of Water Resource, the historically highest groundwater level underlying the site and site vicinity is estimated to be 902 which is more than 30 feet below existing ground surface (see Figures 5, 6 and 7).

10.0 INFILTRATION TEST FOR INFILTRATION RATE

Infiltration tests were performed for soil in borings B-3, B-4, B-5, B-9 and B-10 to evaluate the infiltration rate of onsite soil. The infiltration tests were performed in general accordance with the San Bernardino County "Technical Guidance Document for Water Quality Management Plan (WQMP)". The "Shallow Percolation Test Method" was used in the infiltration test.

Water Data Library

Use the map below to locate monitoring stations. You can find an area of interest if you zoom and pan the map. Use the search gox below to find features on the map such as the name of a city, park, landmark, lake, water feature, or zip code within California. Additional searches by data type are possible by clicking the links on the left. For help on these and other ways to find your data click here.

WDL STATION MAP

Location Search

To find monitoring stations for a specific area, enter the placename or zip code into the text box below

Site Type

Select the desired site type using the checkboxes

- ☑ Groundwater Level
- ☐ Water Quality
  - ☐ Include Historic Data
- ☐ Continuous Data

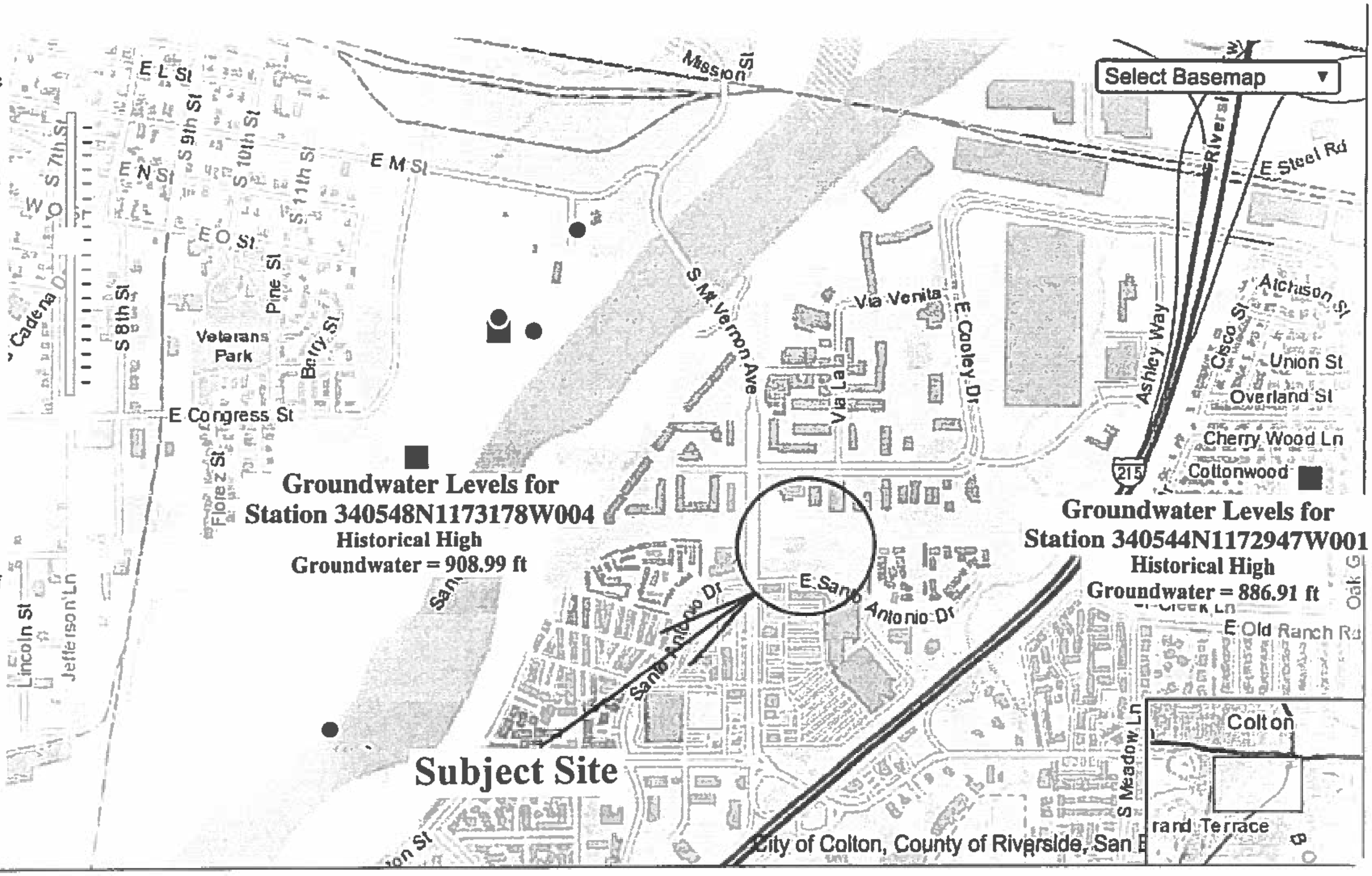
■ = Multiple Stations at one Location

● = Cluster, showing number of stations

Cursor Coordinates (WGS84)

Lat: 34.0624, Long: -117.3029

0 0.15 0.3mi



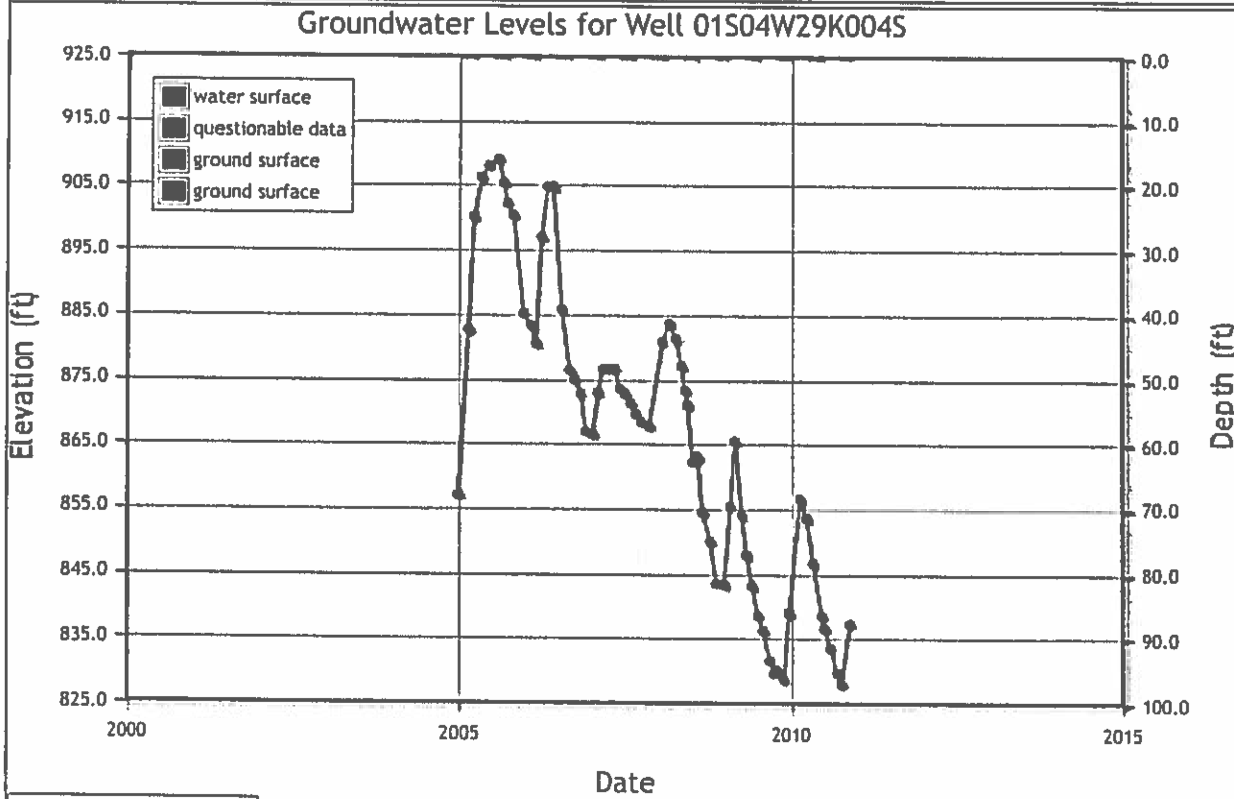
Elevation of Site = 933 ft
Elevation of Historical High Groundwater Estimated = 900 ft

Figure 5

Groundwater Levels for Station 340548N1173178W004

Data for your selected well is shown in the tabbed interface below. To view data managed in the updated WDL tables, including data collected under the CASGEM program, click the "Recent Groundwater Level Data" tab. To view data stored in the former WDL tables, click the "Historical Groundwater Level Data" tab. To download the data in CSV format, click the "Download CSV File" button on the respective tab. Please note that the vertical datum for "recent" measurements is NAVD88, while the vertical datum for "historical" measurements is NGVD29. To change your well selection criteria, click the "Perform a New Well Search" button.

Station Data» Recent Groundwater Level Data» **Historical Groundwater Level Data**



Download CSV File

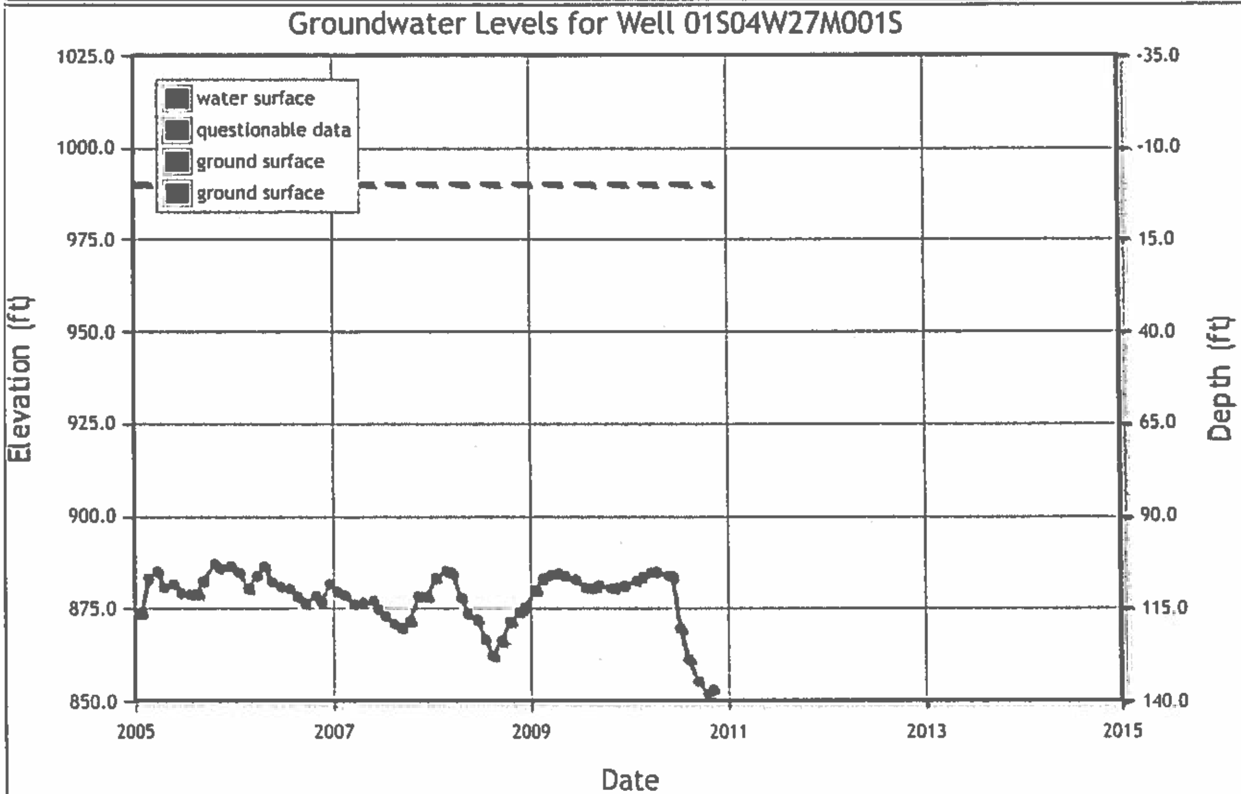
Date	RPE	GSE	RPWS	WSE	GS to WS	NM Code	QM Code	Agency	Comments
12/22/2004 00:00	925	925	68	857	68			5000	
02/16/2005 00:00	925	925	42.39	882.61	42.39			5000	
03/15/2005 00:00	925	925	24.9	900.1	24.9			5000	
04/27/2005 00:00	925	925	18.77	906.23	18.77			5000	
06/07/2005 00:00	925	925	16.99	908.01	16.99			5000	
07/27/2005 00:00	925	925	16.01	908.99	16.01	← **Historical high groundwater**		5000	
08/24/2005 00:00	925	925	19.61	905.39	19.61			5000	
09/13/2005 00:00	925	925	22.73	902.27	22.73			5000	
10/18/2005 00:00	925	925	24.68	900.32	24.68			5000	
12/15/2005 00:00	925	925	39.85	885.15	39.85			5000	
01/26/2006 00:00	925	925	41.75	883.25	41.75			5000	
02/23/2006 00:00	925	925	44.53	880.47	44.53			5000	
03/23/2006 00:00	925	925	28.05	896.95	28.05			5000	
04/26/2006 00:00	925	925	20.21	904.79	20.21			5000	
05/25/2006 00:00	925	925	20.15	904.85	20.15			5000	

Figure 6

Groundwater Levels for Station 340544N1172947W001

Data for your selected well is shown in the tabbed interface below. To view data managed in the updated WDL tables, including data collected under the CASGEM program, click the "Recent Groundwater Level Data" tab. To view data stored in the former WDL tables, click the "Historical Groundwater Level Data" tab. To download the data in CSV format, click the "Download CSV File" button on the respective tab. Please note that the vertical datum for "recent" measurements is NAVD88, while the vertical datum for "historical" measurements is NGVD29. To change your well selection criteria, click the "Perform a New Well Search" button.

Station Data» Recent Groundwater Level Data» **Historical Groundwater Level Data**



Download CSV File

Date	RPE	GSE	RPWS	WSE	GS to WS	NM Code	QM Code	Agency	Comments
01/05/2005 00:00	990	990	116.8	873.2	116.8			5000	
01/28/2005 00:00	990	990	116.24	873.76	116.24			5000	
02/24/2005 00:00	990	990	107.07	882.93	107.07			5000	
03/26/2005 00:00	990	990	105.2	884.8	105.2			5000	
04/18/2005 00:00	990	990	109.41	880.59	109.41			5000	
05/25/2005 00:00	990	990	108.73	881.27	108.73			5000	
06/23/2005 00:00	990	990	111.09	878.91	111.09			5000	
07/27/2005 00:00	990	990	111.43	878.57	111.43			5000	
08/23/2005 00:00	990	990	111.16	878.84	111.16			5000	
09/13/2005 00:00	990	990	107.76	882.24	107.76			5000	
10/26/2005 00:00	990	990	103.09	886.91	103.09	←**Historical high groundwater**		5000	
11/15/2005 00:00	990	990	104.39	885.61	104.39			5000	
12/21/2005 00:00	990	990	103.79	886.21	103.79			5000	
01/20/2006 00:00	990	990	105.49	884.51	105.49			5000	
02/21/2006 00:00	990	990	109.86	880.14	109.86			5000	

Figure 7

The following procedures were followed in the percolation tests: 1) 8-inch diameter holes were drilled to the estimated bottom elevation of proposed infiltration facility, 2) place 2 inches of gravels on the bottom of test holes, 3) presoak the test holes a minimum of twice, 4) add water to perform tests and taking readings of water levels using an interval of 10 minutes, 6) repeat step (5) a minimum of 6 times.

The average drop of the stabilized rate [ΔH (inch) / Δt (minute) x 60] over the last 3 consecutive readings is considered as the pre-adjusted infiltration rate. Where ΔH is the drop of hydraulic head and Δt is the elapsed time. The pre-adjusted infiltration rate is further reduced by times the pre-adjusted infiltration rate by a reduction factor which equal to $r / (r + 2 H_{avg})$. Where r is the radius of test hole and H_{avg} is the average hydraulic head during the test.

The field data of infiltration tests and the calculations of infiltration rate are presented in Appendix C. The infiltration test indicated an infiltration rate in the range of 10 to 12 inch/hour. An infiltration rate of 5 inch/hour is recommended for the design of the infiltration facility. A minimum factor of safety of 2.0 is applied in the determination of he recommended infiltration rate of 5.4 inch/hour.

If the use of a standard infiltration pit for the subject project is accepted by the City of Colton, an infiltration rate of 5.4 inch/hour can be used in the design of the infiltration facility. The bottom of infiltration pit should be a minimum of 10 feet above groundwater. The infiltration pit should be located a minimum of 10 feet horizontally away from the building structure and private property line unless justified by the project soil engineer and approved by the City Engineer.

11.0 FAULTING AND SEISMICITY STUDY

Limited faulting study and seismicity study were performed to evaluate the potential of earthquake-induced hazards which may affect the stability of the site and the building structures on the site. The seismic factors for the structural design of the proposed three-story automobile building were also determined.

11.1 Faulting Study

The faulting study indicated that 22 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 Glen Helen - Lytle Creek Ridge - Claremnont fault, San Gorgonio - Banning fault, San Andreas fault (San Bernardino Mountain) and Cucamonga fault. 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. 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.

11.2 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 0.83g (g:gravity) and 0.74g, respectively. The largest credible and probable repeatable high ground accelerations ($m + \sigma$) which may impact the site are 0.54g and 0.48g, respectively. The mean peak high ground accelerations (PGAm) is 1.044g. The maximum credible magnitude, peak ground acceleration, repeatable high ground acceleration which may impact the site caused by the most significant fault systems and San Andrea fault 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
Glen Helen - Lytle Creek Ridge - Claremont fault	2	7	0.75g	0.49g
San Gorgonio -banning fault	7	7.5	0.83g	0.54g
San Andreas fault (San Bernardino Mountain)	13	8.0	0.60	0.39g
Cucamonga fault	19	7.0	0.39g	0.25g
North Frontal fault zone	25	7.7	0.36g	0.23g
San Andreas fault (San Bernardino Mountain)	30	8.3	0.40	0.26g

11.3 Seismic Factors

The seismic factors listed in the following table can be used in structural design of the proposed automobile building. The seismic factors were determined based on the findings of field exploration and in accordance with ASCE7-16 and 2019 California Building Code (see Figure 8).

Seismic Factors	Value	Reference
Site Class	D	Chapter 20 of ASCE 7
Mapped SRA at 0.2 Second Period (S_S)	2.251g	Figure 1613.3.1 (1) / CBC
Mapped SRA at 1.0 Second Period (S_1)	0.899g	Figure 1613.3.1 (2) / CBC
Site Coefficient Fa	1.0	Table 1613.3.3 (1) / CBC
Site Coefficient Fv	Null	Table 1613.3.3 (2) / CBC
Maximum Considered Earthquake SRA at 0.2 Second Period (Sms)	2.251g	Equation 16-37 / CBC
Maximum Considered Earthquake SRA at 1.0 Second Period (S_{m1})	Null	Equation 16-38 / CBC
Design SRA at 0.2 Second Period (Sds)	1.501	Equation 16-39 / CBC
Design SRA at 1.0 Second Period (S_{d1})	Null	Equation 16-40 / CBC
Seismic Design Category	Null	Chapter 20 of ASCE 7

SRA : Spectral Response Acceleration

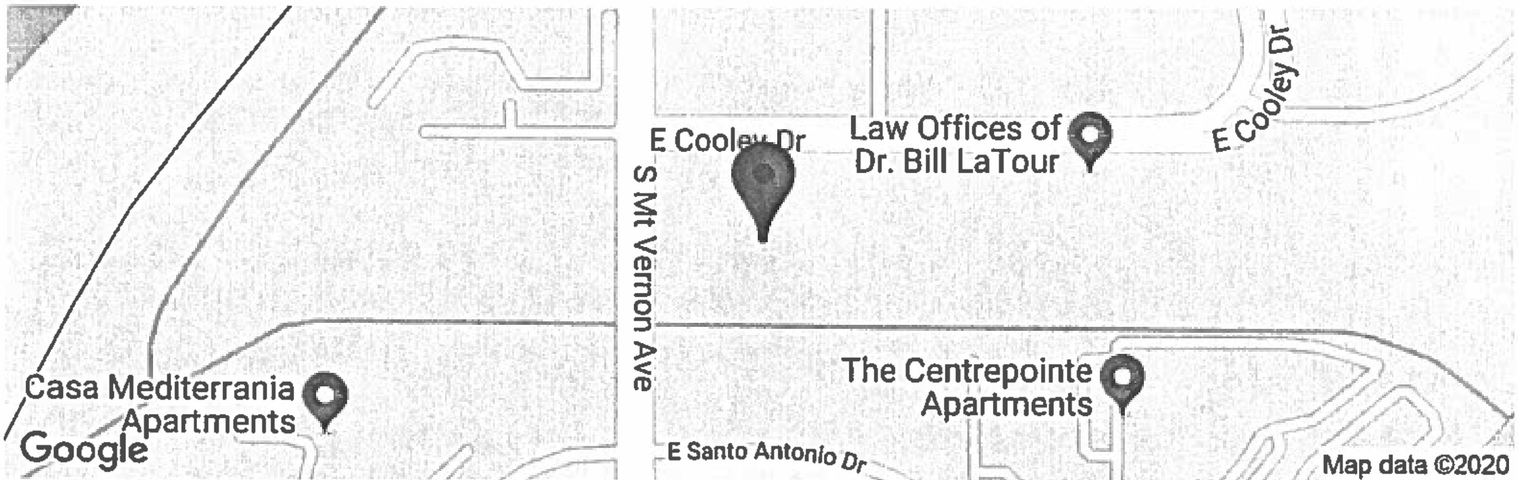
11.4 Earthquake-Induced Geologic Hazards

Based on the findings of field exploration, faulting study, seismicity study and liquefaction evaluation, it is our opinion that the occurrence of earthquake-induced geologic hazards such as lurching, landslide and liquefaction within the site is unlikely. Onsite soils may be susceptible to minor earthquake-induced settlement. 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 earthquake-induced geologic hazards such as liquefaction, ground rupture, landslides, seiches, tsunamis, lurching, and seismically induced settlement are discussed in the following subsections.

OSHPD

Latitude, Longitude: 34.05377024, -117.30799279



Date	30/01/2020, 16:17:08
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	2.251	MCE_R ground motion. (for 0.2 second period)
S_1	0.899	MCE_R ground motion. (for 1.0s period)
S_{MS}	2.251	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.501	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.949	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	1.044	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	2.357	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.579	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.251	Factored deterministic acceleration value. (0.2 second)
S1RT	0.938	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	1.054	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.899	Factored deterministic acceleration value. (1.0 second)
PGAd	0.949	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.914	Mapped value of the risk coefficient at short periods
C_{R1}	0.891	Mapped value of the risk coefficient at a period of 1 s

Figure 8

11.4.1 Liquefaction Evaluation

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 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 50 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 9, the site is located within the Generalized Liquefaction susceptible zones as mapped in the San Bernardino County Land Use Plan. Therefore, a detailed liquefaction evaluation is pwerformed for the subject project.

The liquefaction evaluation method introduced by Seed and Idriss (1985) was used in the calculation of the factors 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".

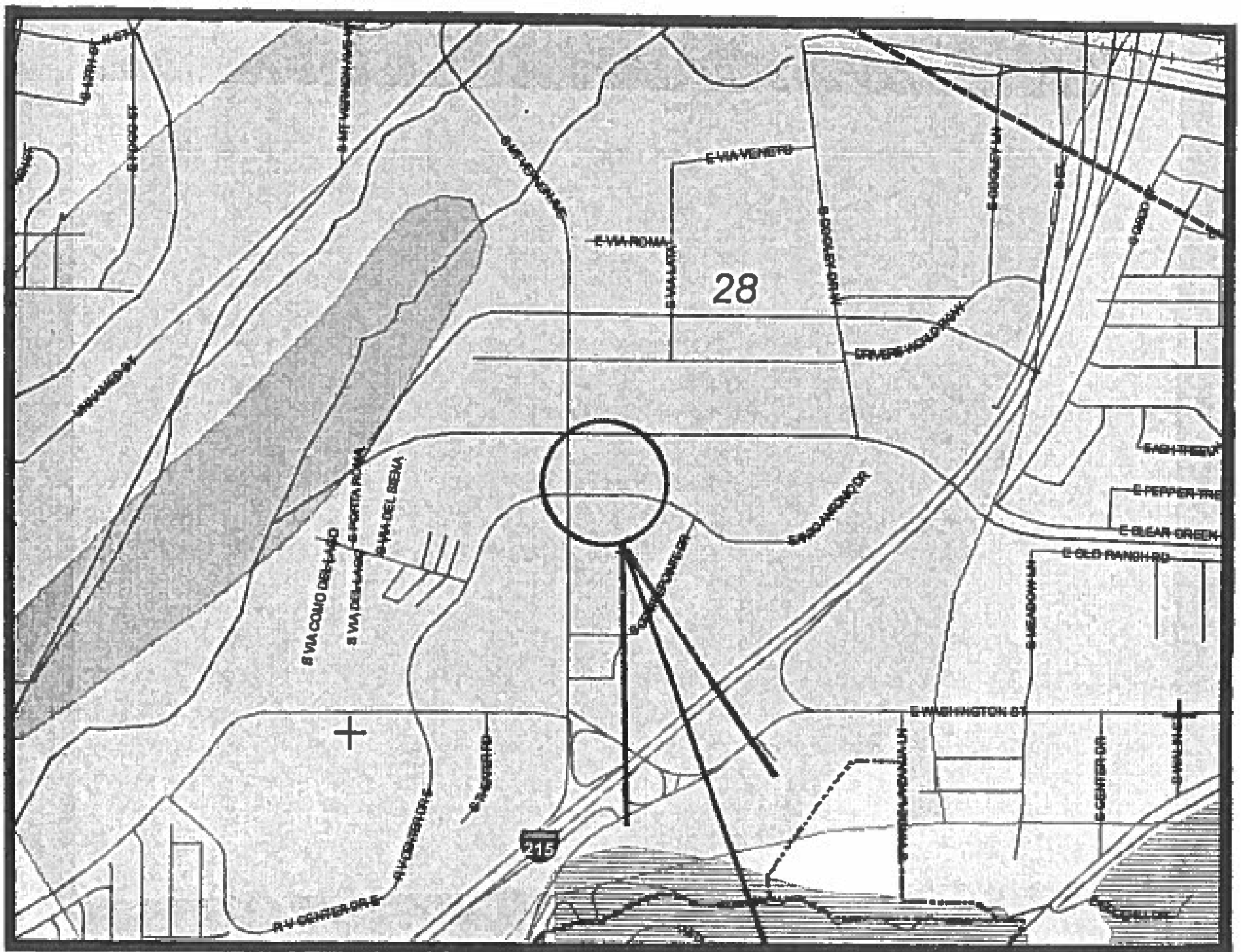
An earthquake magnitude of 7.07 based on the ASCE7-16 and a peak ground acceleration of 0.696g which equal to 2/3 PGAm (mean peak high ground acceleration) were used in liquefaction evaluations. Although no groundwater was encountered at the depth of both deep borings, the historically highest groundwater underlying the site and in site vicinity is estimated to be approximately 30 feet below the ground surface as discussed in Section 9.0. Therefore, a groundwater at a depth of 50 feet below the existing ground surface was used in the calculation of relative density, C_N value and corrected N value ($N_{1(60)}$), and a groundwater depth of 30 feet was used to calculate the earthquake induced stress ratio.

The historically highest groundwater levels, earthquake magnitudes and peak ground accelerations used in the liquefaction evaluations are listed in the following table.

Depth of Groundwater Observed in Boring, ft	Depth to Historically Highest Groundwater, ft	Maximum Credible Magnitude of Earthquake	Peak Horizontal Ground Acceleration PGAm
> 50	30	7.07	0.696g

The input data and calculations of factors of safety for liquefaction potential are shown on Tables B.1 to B.8 in Appendix B. The liquefaction evaluation indicated that onsite soil is not susceptible to liquefaction during an earthquake with the assumed earthquake magnitude, ground accelerations and groundwater level. The results of the liquefaction evaluations are summarized in the following table.

Depth (ft)	Susceptibility to Liquefaction	Remarks
0 - 30	Not Susceptible	Above Groundwater
30 - 50	Not Susceptible	Relatively High SPT N Values



Subject Site

Legend

Generalized Liquefaction Susceptibility
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

Generalized Landslide Susceptibility
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

N

Scale 1 = 14,400

(Adopted from San Bernardino County Land Use Plan)

C. Y. GEOTECH, INC. Engineering Geology and Geotechnical Engineering	**Geologic Hazard Overlays**
	CYG-19-8863 / **Figure 9**

11.4.2 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 11.1, 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 crosses the site. Therefore, the potential of ground rupture or cracking due to shaking from local seismic events is considered to be low.

11.4.3 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 fairly level and not susceptible to liquefaction. Therefore, the site is not subject to earthquake-induced landsliding or lateral spreading.

11.4.4 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 flat and onsite soils are competent. Therefore, the potential of the occurrence of ground lurching with the site is considered low. Suitable site processing can eliminate compressible materials of low relative density and, thereby, will tend to reduce the potential for ground lurching.

11.4.5 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.

11.4.6 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 ground acceleration, the strength of soils and the occurrence of groundwater. It is our opinion that the potential of damage caused by earthquake-induced settlement after the completion of the proposed building will be low because onsite soils consists of competent silty sand, sand and gravelly sand and the foundation for building support will be founded into compacted fill.

12.0 SLOPE STABILITY

The site is fairly level and, therefore, not susceptible to any type of landslide. No evidence of deep-seated slope failure or other type of slope failure was observed within the site during site observation. The site is not located within any of the landslide areas mapped in the available public geologic and geotechnical maps. As shown on Figure 9, the site is not located with any of the earthquake-induced landslide zones as mapped in the CDMG seismic hazard maps.

Wedge slope stability analyses and slot cut calculations were performed to evaluate the stability of temporary excavations.

12.1 Wedge Slope Stability Analysis

Wedge slope stability analyses using the Free Body Diagram method were performed to evaluate the stability of a 2-foot high vertical cut and 10-foot high temporary excavation with the lower 1 foot vertical and the upper 9 feet 1:1 trimming. The peak shear strength parameters of alluvial soil were used in the analyses. The analyses indicated factor of safety greater than the minimum code requirement for both cases. The results of the analyses are shown in Figures 10 and 11.

12.2 Slot Cut Calculation

A slot cut calculation was performed to evaluate the stability of a 5-foot high and 3-foot wide A/B/C slot cut. The peak shear strength parameters of alluvial soil were used in the calculation. The calculation indicated a factor of safety greater than the minimum code requirement. The results of the calculation are shown in Figure 12.

13.0 SETTLEMENT EVALUATION

The potential of foundation settlements of the proposed automobile office building were evaluated. The settlement evaluations were performed based on the structural load and foundation dimensions assumed in the following table. Additional foundation settlement evaluations can be performed, if requested, after the actual structural load and foundation dimensions are designed by the Project Structural Engineer.

Settlement of Square Footings Caused by Static Structural Load

Foundation Type	Dimension	Embedment Depth	Applied Stress (Static + Seismic), psf	Total Foundation Settlement D (static + seismic), inch
Square Footing	2 ft x 2 ft	2 ft	2000	0.34
Square Footing	3 ft x 3 ft	2 ft	2400	0.57
Square Footing	4 ft x 4 ft	2 ft	2800	0.91
Square Footing	5 ft x 5 ft	2 ft	3200	1.23

Settlement of Continuous Footings Caused by Static Structural Load

Foundation Type	Dimension	Embedment Depth	Applied Stress (Static + Seismic), psf	Total Foundation Settlement D (static + seismic), inch
Continuous Footing	1.5 ft wide	2 ft	2000	0.39
Continuous Footing	2 ft wide	2 ft	2200	0.56
Continuous Footing	3 ft wide	2 ft	2600	1.03
Continuous Footing	4 ft wide	2 ft	3000	1.74

Settlement of Square Footings Caused by Static Structural Load Plus Vertical Earthquake Shaking

Foundation Type	Dimension	Embedment Depth	Applied Stress (Static + Seismic), psf	Total Foundation Settlement D (static + seismic), inch
Square Footing	2 ft x 2 ft	2 ft	2000 + 1200	0.49
Square Footing	3 ft x 3 ft	2 ft	2400 + 1440	0.82
Square Footing	4 ft x 4 ft	2 ft	2800 + 1680	1.22
Square Footing	5 ft x 5 ft	2 ft	3200 + 1920	1.69

Equivalent Fluid Pressure (Free Body Diagram Method)

Program Made by C. Y. Geotech, Inc. (Version 17.0)

Project Name:

CYG-19-8863 2 feet Temporary Cut / Level (Soil)

GEOMETRY OF CRITICAL ACTIVE WEDGE:

Height of the Temporary Cut	=	2 feet
Slope Angle of Retained Slope	=	0 degree
Dip Angle of Critical Wedge	=	57 degree

SHEAR STRENGTH PARAMETERS:

Unit Weight	=	104 pcf
Cohesion	=	60 psf
Friction Angle	=	30 degree
Mobilized Cohesion	=	48 psf
Mobilized Friction Angle	=	24.8 degree

REQUIRED FACTOR OF SAFETY = 1.25

RESULTS

Dip Angle of Critical Slip Surface	=	57 degree
Total Weight of Active Wedge	=	135 lbs
Frictional Resistance (Cm * L)	=	114 lbs
Required External Force for FS = 1.25	=	-38 lbs
Required Equivalent Fluid Pressure	=	-18.9 psf/ft

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

Figure 10

Equivalent Fluid Pressure (Free Body Diagram Method)

Program Made by C. Y. Geotech, Inc. (Version 17.0)

Project Name:

CYG-19-8863 10'-high Temporary Cut with the Lower 1'-high Verticial and Upper 9'-high 1:1 Trimming (Soil)

GEOMETRY OF CRITICAL ACTIVE WEDGE:

Height of the Temporary Cut	=	1 feet
Height of the Slope Above Cut	=	9 feet
Slope Angle of Retained Slope	=	45 degree
Dip Angle of Critical Wedge	=	38 degree

SHEAR STRENGTH PARAMETERS:

Unit Weight	=	104 pcf
Cohesion	=	60 psf
Friction Angle	=	30 degree
Mobilized Cohesion	=	48 psf
Mobilized Friction Angle	=	24.8 degree

REQUIRED FACTOR OF SAFETY = 1.25

Change of Weight for Irregular Geometry	=	0 lbs
Additional Lateral Resistance From Front Wedge	=	0 lbs

RESULTS

Dip Angle of Critical Slip Surface	=	38 degree
Total Weight of Active Wedge	=	2444 lbs
Frictional Resistance (Cm * L)	=	780 lbs
Required External Force for Wall	=	-153 lbs
Required Equivalent Fluid Pressure	=	-307 psf/ft

Figure 11

5 ft high / 3 ft wide / 0 lbs/ft surcharge / Soil / A-B-C Slot Cut

Program Made by C. Y. Geotech, Inc. (Version 17.0)

CYG-19-8863 **Minimum Factor of Safety = 1.3**

Height	Spacing	Surcharge	Unit Wt.	Cohesion	Friction Angle	Delta	Length	Weight	Sliding Force	RF1	RF2	RF3	FS
H = ft	S = ft	q = lbs/ft	pcf	C = psf	ϕ = degree	δ = degree	L = ft	W = lbs	SF = lbs	lbs	lbs	lbs	
5	3	0	104	60	30	55	6.1	2731	2237	904	1099	1050	1.36
5	3	0	104	60	30	56	6.0	2631	2181	849	1086	1012	1.35
5	3	0	104	60	30	57	6.0	2533	2124	796	1073	974	1.34
5	3	0	104	60	30	58	5.9	2437	2067	746	1061	937	1.33
5	3	0	104	60	30	59	5.8	2343	2009	697	1050	901	1.32
5	3	0	104	60	30	60	5.8	2252	1950	650	1039	866	1.31
5	3	0	104	60	30	61	5.7	2162	1891	605	1029	831	1.30
5	3	0	104	60	30	62	5.7	2074	1831	562	1019	798	1.30
5	3	0	104	60	30	63	5.6	1987	1771	521	1010	764	1.30
5	3	0	104	60	30	64	5.6	1902	1710	481	1001	732	1.30
5	3	0	104	60	30	65	5.5	1819	1648	444	993	699	1.30
5	3	0	104	60	30	66	5.5	1736	1586	408	985	668	1.30
5	3	0	104	60	30	67	5.4	1655	1524	373	978	637	1.30
5	3	0	104	60	30	68	5.4	1576	1461	341	971	606	1.31
5	3	0	104	60	30	69	5.4	1497	1398	310	964	576	1.32
5	3	0	104	60	30	70	5.3	1419	1334	280	958	546	1.34
5	3	0	104	60	30	71	5.3	1343	1270	252	952	516	1.36
5	3	0	104	60	30	72	5.3	1267	1205	226	946	487	1.38
5	3	0	104	60	30	73	5.2	1192	1140	201	941	459	1.40
5	3	0	104	60	30	74	5.2	1118	1075	178	936	430	1.44
5	3	0	104	60	30	75	5.2	1045	1009	156	932	402	1.48

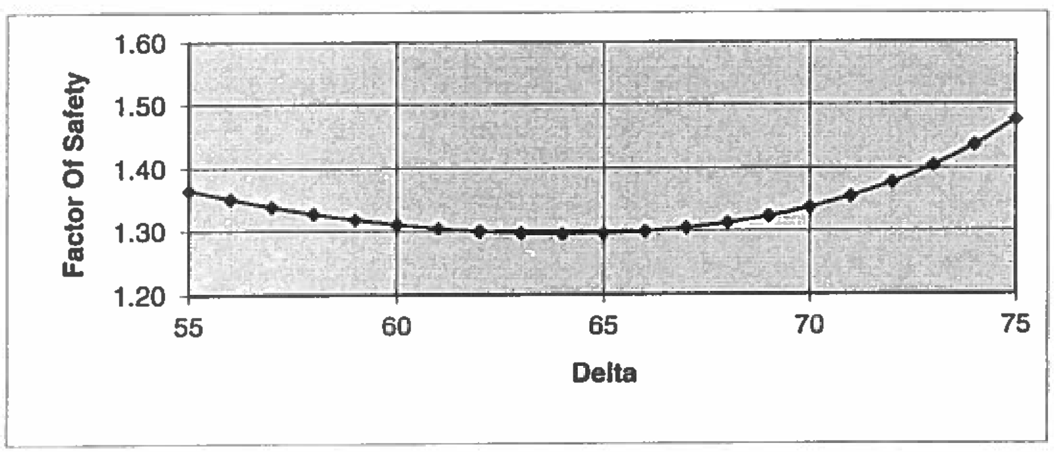


Figure 12

Settlement of Continuous Footings Caused by Static Structural Load Plus Vertical Earthquake Shaking

Foundation Type	Dimension	Embedment Depth	Applied Stress (Static + Seismic), psf	Total Foundation Settlement D (static + seismic), inch
Continuous Footing	1.5 ft wide	2 ft	2000 + 1200	0.63
Continuous Footing	2 ft wide	2 ft	2200 + 1320	0.95
Continuous Footing	3 ft wide	2 ft	2600 + 1560	1.60
Continuous Footing	4 ft wide	2 ft	3000 + 1800	2.45

Settlement of Square Footings Caused by Vertical Earthquake Shaking

Foundation Type	Dimension	Embedment Depth	Applied Stress Seismic, psf	Earthquake-Induced Foundation Settlement D (seismic), inch
Square Footing	2 ft x 2 ft	2 ft	from 2000 to 3200	0.15
Square Footing	3 ft x 3 ft	2 ft	from 2400 to 3840	0.25
Square Footing	4 ft x 4 ft	2 ft	from 2800 to 4480	0.31
Square Footing	5 ft x 5 ft	2 ft	from 3200 to 5120	0.46

Settlement of Continuous Footings Caused by Vertical Earthquake Shaking

Foundation Type	Dimension	Embedment Depth	Applied Stress Seismic, psf	Earthquake-Induced Foundation Settlement D (seismic), inch
Continuous Footing	1.5 ft wide	2 ft	from 2000 to 3200	0.24
Continuous Footing	2 ft wide	2 ft	from 2200 to 3520	0.39
Continuous Footing	3 ft wide	2 ft	from 2600 to 4160	0.57
Continuous Footing	4 ft wide	2 ft	from 3000 to 4800	0.71

Due to the sandy and gravelly nature of onsite soils, it is estimated that more than 50% of the total and differential foundation settlement calculated will be occurred prior to the completion of the construction. Total settlement and differential settlement of the proposed automobile office building after the completion of construction supported by conventional spread footings as recommended are anticipated to be within tolerable limits. Total foundation settlement of the automobile office building after the completion of construction is expected to be less than 0.9 inch for static structural load and 0.7 inch for seismic loading. Differential foundation settlement of the automobile office building after the completion of construction is estimated to be less than 0.7 inch for static structural load and 0.5 inch for seismic loading.

14.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the findings of this investigation, the development of the proposed automobile office building and recreation vehicle parking lot is feasible from soils engineering viewpoints provided the recommendations in this report are properly incorporated into design and implemented during construction. The recommendations in the following subsections should be incorporated into the design of the proposed automobile office building and implemented during construction. Conventional spread footing founded into compacted fill can be used to support the proposed automobile building. The asphalt concrete pavement should be supported by a minimum of two feet of compacted fill. An infiltration rate of 5.4 inches per hour can be used in the design of infiltration system.

14.1 Foundation System

Conventional spread footing founded into compacted fill can be used to support the proposed automobile office building. Soil removal and recompaction of will be required to create a competent building pad for building support.

14.2 Site Preparation

The upper 5 feet of onsite soils, at their present conditions, are not suitable for foundation support. Soils removal and recompaction will be required if conventional spread footings and/or concrete slabs-on-grade supported by compacted fill are proposed. All fill placement and soil compaction should be performed in conformance with the current grading ordinances of the City of Colton. The following recommendations should be incorporated into the design and implemented during construction.

a. The existing soil in the building pad area should be removed to a minimum of 5 feet below the existing ground surface or a minimum of 3 feet below the bottom of the footings, whichever is deeper, and then recompacted to be compacted fill for building support. The removal and recompaction should be extended horizontally to a minimum of 3 feet beyond the perimeter footings in all directions. The removal and recompaction can be limited to property lines.

b. The existing soil in the interior concrete slab area should be removed to a minimum of 2 feet below the existing ground surface or a minimum of 1 foot below the bottom of concrete slab, whichever is deeper, and then recompacted to be compacted fill for slab support. The removal and recompaction can be limited to surrounding footings.

c. If cement concrete driveway supported by compacted fill is proposed, the existing soil in the cement concrete driveway area should be removed to a minimum of 2 feet below the existing ground surface or a minimum of 1 foot below the bottom of cement concrete driveway, whichever is deeper, and then recompacted to be compacted fill for driveway support. The removal and recompaction should be extended horizontally to a minimum of 2 feet beyond the boundaries of cement concrete driveway in all directions. The removal and recompaction can be limited to property lines and adjacent building structures.

d. If asphalt concrete pavement supported by compacted fill is proposed, the existing soil in the asphalt concrete pavement area should be removed to a minimum of 2 feet below the existing ground surface or a minimum of 1 foot below the bottom of asphalt concrete pavement, whichever is deeper, and then recompacted to be compacted fill for pavement support. The removal and recompaction should be extended horizontally to a minimum of 1 foot beyond the boundaries of asphalt concrete pavement in all directions. The removal and recompaction can be limited to property lines and adjacent building structures.

e. If exterior concrete slab-on-grade supported by compacted fill is proposed, the existing soil in the exterior concrete slab area, except the cement concrete driveway area, should be removed to a minimum of 2 feet below the existing ground surface or a minimum of 1 foot below the bottom of the exterior concrete slab, whichever is deeper, and then recompacted to be compacted fill for slab support. The removal and recompaction should be extended horizontally to a minimum of 1 foot beyond the boundaries of exterior concrete slab in all directions. The removal and recompaction can be limited to property lines and adjacent building structures.

f. If deep loose soil was found at bottom recommended to support fill soil, the loose soil should be over-excavated to underlying competent soil as determined by CYG, and then recompacted to be compacted fill for foundation and slab support.

14.3 Conventional Spread Footing

Conventional spread footings founded into compacted fill can be used to support the proposed automobile office building. The following recommendations can be used in the design of conventional spread footings.

a. All conventional spread footings should be vertically supported by compacted fill prepared as recommended in Section 14.2. All conventional spread footings should be provided with a minimum of 3 feet of compacted fill below the bottom of footings.

b. Continuous footings should have a minimum width of 12 inches and a minimum embedment depth of 24 inches into compacted fill. Isolated footings should have a minimum width of 24 inches and a minimum embedment depth of 24 inches into compacted fill.

c. An allowable vertical bearing pressure of 2000 pounds per square foot (psf), including dead and frequently applied live loads, can be used in the design of footings with the minimum footing width and embedment depth. The allowable bearing capacity can be increased by 400 psf for each additional foot of footing width or embedment depth, to a maximum bearing capacity of 4000 psf. The vertical bearing capacities can be increased by one-third (1/3) when considering short duration wind or seismic loads.

d. Lateral force can be resisted by frictional resistance and passive earth pressure. A friction coefficient of 0.35 and an allowable passive earth pressure of 300 pounds per square foot per foot of depth (psf/ft), to a maximum of 1500 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 (1/3).

e. All footings should have a minimum reinforcement of two No.4 steel bars near the top of footing and two No.4 steel bars near the bottom of footings. Where footing 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.

f. Prior to the placement of steel in footing excavations, CYG should be notified to inspect and approve the footing excavations to ensure that the footing excavations expose compacted fill and are free of loose soil or any other deleterious materials.

g. The City Inspector should be notified to inspect and approve the footing excavations prior to pouring concrete.

14.4 Foundation Settlement

The total and differential foundation settlements of the proposed automobile office building supported by conventional spread footings founded into compacted fill as recommended are anticipated to be within tolerable limits.

Total foundation settlement of the proposed automobile office building is expected to be less than 1-1/2 inch after the completion of construction. Differential foundation settlement of the proposed automobile office building is expected to be less than 3/4 inch after the completion of construction. Due to the sandy and gravelly nature of onsite soils, it is estimated that more than 50% of the calculated total and differential foundation settlement discussed in Section 13.0 will be occurred prior to the completion of the construction.

It should be noted that the evaluation of foundation settlement is based on the assumption that the area and surrounding areas of the proposed office building 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 and/or soil lubrication may occur if the foundation soil is saturated or nearly saturated. In order to avoid the migration of a significant amount of surface or subsurface water to foundation soil, the recommendations in the section of "Drainage Control" should be incorporated into the design and implemented during construction. The drainage devices should be constantly maintained.

14.5 Asphalt Concrete Pavement

Asphalt concrete pavement should be entirely supported by compacted fill. If asphalt concrete pavement supported by compacted fill is proposed, the existing soil in the asphalt concrete pavement area should be removed to a minimum depth of 2 feet below the existing ground surface or a minimum of 1 foot below the bottom of the asphalt concrete pavement, whichever is deeper, and then recompacted to be certified fill.

Structural section calculations for asphalt concrete pavement are 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 38 and traffic indexes of 4, 5 and 6 were assumed in the calculations of structural sections. The results of calculations are shown on 13, 14 and 15 and summarized in the following Table. A traffic index of 4 is recommended for area of passenger cars. A traffic index of 5 is recommended for areas of delivery or working zones and light weight truck. A traffic index of 6 is recommended for areas of heavy delivery working zones, heavy truck and heavy recreation vehicles.

Traffic Index	Asphalt Concrete	Aggregate Base
4	2.5	4.0
5	3.0	4.0
6	4.0	6.0

Compaction tests will be required for the aggregate base. A minimum relative compaction of 95% is required for aggregate base. If the asphalt 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.

14.6 Slab-On-Grade

Interior concrete slabs-on-grade should be entirely supported by compacted fill. Otherwise, the interior concrete slabs-on-grade should be designed as structural slabs and supported by surrounding footings recommended for building support. If interior concrete slabs supported by compacted fill are proposed, the existing soils in the interior concrete slab areas should be removed to a minimum of 2 feet below the existing ground surface and then recompacted to be compacted fill for slab support.

Structural slabs should be designed by the Project Structural Engineer. Concrete slabs-on-grade supported by compacted fill should be designed for a minimum thickness of 5 inches, reinforced with No.4 bars at 16 inches on centers, both ways. Reinforcement should be properly supported to assure desired mid-height placement. These parameters for slab reinforcement should be reviewed by the Project Structural Engineer and revised as required to accommodate intended use.

A plastic vapor barrier of 10-mil or thicker should be placed below the floor slabs in moisture sensitive areas. The vapor barrier should be either placed beneath the concrete slab and overlying 4 inches of gravels. The vapor barriers should be properly sealed in the joint areas. If the vapor barrier is to be placed beneath the concrete slab, a low slump concrete should be used for concrete slab to minimize possible damage of vapor barrier caused by curling of concrete slab

Flexible Pavement Design Using California Design Guide

TRAFFIC INDEX = 4

R-VALUES:

AGGREGATE BASE	= 80	
AGGREGATE SUBBASE	= 38	
BASEMENT MATERIAL	= 38	

GRAVEL EQUIVALENT REQUIRED:

ASPHALT CONCRETE	= 3.1	INCHES
AGGREGATE BASE	= 9.5	INCHES
AGGREGATE SUBBASE	= 9.5	INCHES

GRAVEL EQUIVALENT FACTOR:

ASPHALT CONCRETE	= 2.83
AGGREGATE BASE	= 1.1
AGGREGATE SUBBASE	= 1.0

MINIMUM THICKNESS DESIGN:

ASPHALT CONCRETE	= 1.1	INCHES
AGGREGATE BASE	= 5.8	INCHES
AGGREGATE SUBBASE	= 0	INCHES

FULL DEPTH ASPHALT CONCRETE DESIGN:

ASPHALT CONCRETE	= 3.4	INCHES

Proposed Pavement Design

DESIGN THICKNESS OF ASPHALT CONCRETE	= 2.5	INCHES
DESIGN THICKNESS OF AGGREGATE BASE	= 4	INCHES
DESIGN THICKNESS OF AGGREGATE SUBBASE	= 0	INCHES
TOTAL GRAVEL EQUIVALENT REQUIRED	= 9.5	INCHES
DESIGN TOTAL GRAVEL EQUIVALENT	= 11.5	INCHES

DESIGN GRAVEL EQUIVALENT > REQUIRED GRAVEL EQUIVALENT

C. Y. GEOTECH, INC. Engineering Geology and Geotechnical Engineering	**Pavement Design Calculation** **Traffic Index = 4**
CYG-19-8863	**Figure 13**

Flexible Pavement Design Using California Design Guide

TRAFFIC INDEX = 5

R-VALUES:

AGGREGATE BASE	= 80	
AGGREGATE SUBBASE	= 38	
BASEMENT MATERIAL	= 38	

GRAVEL EQUIVALENT REQUIRED:

ASPHALT CONCRETE	= 3.8	INCHES
AGGREGATE BASE	= 11.9	INCHES
AGGREGATE SUBBASE	= 11.9	INCHES

GRAVEL EQUIVALENT FACTOR:

ASPHALT CONCRETE	= 2.53	
AGGREGATE BASE	= 1.1	
AGGREGATE SUBBASE	= 1.0	

MINIMUM THICKNESS DESIGN:

ASPHALT CONCRETE	= 1.5	INCHES
AGGREGATE BASE	= 7.4	INCHES
AGGREGATE SUBBASE	= 0	INCHES

FULL DEPTH ASPHALT CONCRETE DESIGN:

ASPHALT CONCRETE	= 4.7	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	= 11.9	INCHES
DESIGN TOTAL GRAVEL EQUIVALENT	= 12	INCHES

DESIGN GRAVEL EQUIVALENT > REQUIRED GRAVEL EQUIVALENT

C. Y. GEOTECH, INC. Engineering Geology and Geotechnical Engineering	**Pavement Design Calculation** **Traffic Index = 5**
	CYG-19-8863 / **Figure 14**

Flexible Pavement Design Using California Design Guide

TRAFFIC INDEX = 6

R-VALUES:

AGGREGATE BASE	= 80	
AGGREGATE SUBBASE	= 38	
BASEMENT MATERIAL	= 38	

GRAVEL EQUIVALENT REQUIRED:

ASPHALT CONCRETE	= 4.6	INCHES
AGGREGATE BASE	= 14.3	INCHES
AGGREGATE SUBBASE	= 14.3	INCHES

GRAVEL EQUIVALENT FACTOR:

ASPHALT CONCRETE	= 2.31
AGGREGATE BASE	= 1.1
AGGREGATE SUBBASE	= 1.0

MINIMUM THICKNESS DESIGN:

ASPHALT CONCRETE	= 2	INCHES
AGGREGATE BASE	= 8.8	INCHES
AGGREGATE SUBBASE	= 0	INCHES

FULL DEPTH ASPHALT CONCRETE DESIGN:

ASPHALT CONCRETE	= 6.2	INCHES

Proposed Pavement Design

DESIGN THICKNESS OF ASPHALT CONCRETE	= 4	INCHES
DESIGN THICKNESS OF AGGREGATE BASE	= 6	INCHES
DESIGN THICKNESS OF AGGREGATE SUBBASE	= 0	INCHES
TOTAL GRAVEL EQUIVALENT REQUIRED	= 14.3	INCHES
DESIGN TOTAL GRAVEL EQUIVALENT	= 15.8	INCHES

DESIGN GRAVEL EQUIVALENT > REQUIRED GRAVEL EQUIVALENT

C. Y. GEOTECH, INC. Engineering Geology and Geotechnical Engineering	Pavement Design Calculation Traffic Index = 6
CYG-19-8863	Figure 15

Concrete decking, slabs and walkways are likely to experience cracking as the results of the curing process of the concrete. The occurrence, amount and locations of shrinkage cracks can be affected to a major and minor degree by the following factors: type of cement, type of aggregate, mix proportions, concrete strength, steel reinforcement, methods of placing and curing, temperature and humidity. 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.

It should be noted that concrete slabs can be uneven to a major and minor degree caused by the following reasons: 1) built-in deflection caused by construction, 2) static soil settlement caused by structural loading, 3) additional static soil settlement and swelling caused by saturation of foundation soil, and 4) earthquake-induced settlement. The built-in deflection will not cause the distress of concrete slab after the construction of concrete slab. However, the deflections caused by static settlement, earthquake-induced settlement may cause the occurrence of new cracks and degradation of shrinkage cracks.

Concrete slabs should have proper water ratio and sand/gravel ratio. The bearing subgrade of concrete slab should not be significantly disturbed during the placement of steel reinforcement and vapor barrier. Non-shrinkage cement can be used to minimize the occurrence of shrinkage cracks. The degradation of shrinkage cracks and the occurrence of new cracks can be minimized by providing the site with a proper surface and subsurface drainage control, including the design and maintenance of drainage system.

14.7 Fill Placement and Soil Compaction

Fill placement and soil compaction will be required to prepare foundation pad for the proposed automobile office building and the subgrade of recreation vehicle parking lot. All fill placement and soil compaction should be performed in conformance with the City of Colton current grading ordinances. The following general guidelines can be used as a basis for quality control of fill placement and soil compaction.

a. Excavated onsite soils, clean 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.

b. Prior to fill placement, remove loose soil, fill soil, construction debris and all other deleterious materials observed in the fill placement area.

c. The bottom to receive fill soil should be inspected and approved by the representative of CYG and then by the City Inspector prior to placing any fill soil.

d. 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.

e. Compacted fill should be placed in controlled layers which, when compacted, should not exceed 6 inches in thickness. The compaction of a thicker layer may result in the failure of field density test.

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.

g. Both sand cone method and nuclear gauge method will be required for field density test. At least one field density test should be made for every 2 feet of vertical lift.

h. If the test indicates a dry density less than the required compaction degree, the tested layer should be removed, recompacted and retested until a minimum dry density 90% of the maximum dry density is achieved.

CYG should be notified to perform required observation and testing of fill placement and soil compaction. It is the responsibility of you, your representative or your contractor to notify CYG to perform the required fill placement observations and field density tests, and to notify the City Inspector to inspect and approve the bottom to receive fill soil.

A soil compaction report with a certificate for the compliance of fill placement and soil compaction to the City of Colton regulations will be required for the City of Colton to sign off the fill placement and soil compaction. A soil compaction report will be prepared by CYG per your request. The soil compaction report should be submitted to the City of Colton after the completion of fill placement and soil compaction.

The observation of fill placement and the testing of soil density are usually performed based on an on-call basis and not a full-time basis. Therefore, the findings of the compaction report will be based on the assumption that the areas selected for testing are representative. The conditions in the soil compaction report will be based on tests and observations of the grading procedures used and represent our engineering opinion as to the contractor's compliance with the project requirements.

It should be noted that it is your responsibility to notify CYG to perform required fill placement observation and field density test. The CYG's work and responsibilities will not include any management, supervision, direction or scheduling of the actual work of the contractor or the contractor's personnel. The presence of the field representative of CYG at the site is only to perform soil compaction test and to provide the owner professional advice, opinions, and recommendations based upon findings of soil compaction tests.

All lines and grades for the proposed development should be provided by you, the general contractor or the grader. CYG is neither the project grading engineer nor the project surveyor. Therefore, CYG should not be assumed any responsibility for lines and grades. If the locations of the structures to be supported by compacted fill are changed after the completion of fill placement, CYG should be notified immediately for the change. Additional soil removal, fill placement and soil compaction may be required if the locations of the structures to be supported by compacted fill are changed after the completion of fill placement.

<u>14.8 Temporary Excavation</u>

Temporary excavations below the 1:1 line projected downward from the bottom of adjacent structures will be considered as the removal of lateral support from the adjacent structures. Temporary excavations below the 1:1 lines projected downward from the property line will be considered as the removal of lateral support from the adjacent property. Temporary excavation which removes lateral support of any adjacent structure or property should be protected by a shoring system or conducted using A/B/C slot cut method.

The removal of lateral support of adjacent property or structure caused by the temporary excavations recommended for the subject project is not anticipated. Therefore, no recommendations for the design and construction of A/B/C slot cut and shoring system are provided in this report. If A/B/C slot cut or shoring system are required and proposed, the stability of A/B/C slot cut and shoring system will be evaluated by CYG and the recommendations for the design and construction of A/B/C slot cut and shoring system will be provided by CYG.

Temporary excavation deeper than 5 feet, unless deep footing are proposed, is not anticipated for the subject project. Temporary excavation more than 5 feet high in soil will require conventional shoring per CAL/OSHA regulations, or the temporary excavations should be performed in accordance with the recommendations in the following table.

Excavation Height (H), ft	Removal of Lateral Support of Adjacent Structure (Y/N)	Horizontal : Vertical
$H \leq 2$	No	Vertical
$2 < H \leq 10$	No	Vertical for lower 1 foo and 1:1 trimming for above 1 foot
$H > 10$	Yes	Additional engineering evaluations are required

All excavations without shoring 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. No surcharge loading is allowed within the top 5 feet of temporary excavations. Of particular concern is the possibility of heavy construction equipment being placed close to the excavation.

14.9 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. Yard areas and planter areas should be provided with adequate area drains to intercept surface water. Landscape watering should be kept to the minimum amount required for vegetation growth. Building structures should be provided with roof gutters/drains and downspouts. The outlets of downspouts should be 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. Proper drainage should be provided to divert surface water away from the foundation and footing areas during construction. This is especially important when construction takes place during rainy seasons.

It should be noted that the evaluations of foundation settlement for the proposed automobile office building were based on the assumption that the areas and surrounding areas of the proposed automobile office building will be provided with adequate surface drainage devices and that the drainage system will be properly and constantly maintained. Additional settlement caused by local bearing failure and/or soil lubrication may occur if foundation soil is saturated or nearly saturated. In order to avoid the migration of a significant amount of surface water to foundation soil, the recommendations for drainage control in this section should be incorporated into the design and implemented during construction. The drainage devices should be constantly maintained.

The purpose of providing recommendations for drainage control in this section is to remind you, your representative, general contractor and the design engineer of drainage system that the foundation pad and adjacent area should be provided with adequate surface drainage device so that adverse impacts from surface and subsurface water to the stability of building foundation and footings can be avoided or reduced. It should be noted that it is the responsibility of the designer of drainage system to evaluate and design surface and subsurface drainage systems.

The design of drainage system, as well as the inspection and approval of surface and subsurface drainage are beyond the scope of services provided by soils engineers. Therefore, it is strongly recommended that design engineer of drainage system be notified to inspect and approve the final conditions of surface and subsurface drainage system prior to the completion of the project.

You should be aware that it is your responsibility to ensure that the recommendations for drainage control are incorporated into the design and implemented during construction. The home owner should also be aware that it is the responsibility of the home owner to maintain the drainage devices.

15.0 PLAN REVIEW

Foundation plans and grading plans should be reviewed and approved by the project soils engineer to ensure that the recommendations in the geotechnical engineering reports and the requirements in the city approval letters are properly incorporated into the design plans.

Engineering for the proposed project should not be finalized until approval of this report is obtained from the City of Colton as significant changes in the design and recommendations may result from the city review process. Formal plans ready for submittal to the City of Colton should be reviewed by CYG. Any change in scope of the project may require additional work.

16.0 FIELD INSPECTION AND TESTING

The following field inspection and testing should be performed by CYG to ensure that the recommendations in this geotechnical engineering report and the design requirements in the city approved plans are properly implemented during grading and construction.

a. Inspect and approve footing excavations for conventional spread footings.

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

c. Perform field density test for compacted fill.

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.

It should be noted that it is the responsibility of you, your representative or your contractor to perform all required fill placement and soil compaction, to notify CYG to perform the required inspections and field density tests, and to notify the City Inspector to perform the required inspections.

17.0 LIMITS AND LIABILITY

It should be noted that the recommendations in this report can be considered valid only when the design plans for the subject project are reviewed, approved, signed and stamped by CYG and the construction of the proposed site improvements are properly inspected, tested and approved by CYG. The requirements for plan review, field inspection and field testing are discussed in Sections 15.0 and 16.0 of this report. CYG should not be assumed any responsibility for the subject project if the design plans are not reviewed, approved, signed and stamped by CYG or the construction of the proposed development are not inspected, tested and approved by CYG.

The findings, conclusions and recommendations submitted in this geotechnical engineering report are based on the findings of our data research, map review, subsurface exploration, laboratory testing, seismicity study, engineering evaluation and engineering analysis. The nature and extent of variations in subsurface conditions may not become evident until construction. If variations of site conditions were observed during grading and construction, CYG should be notified immediately. If variations appear evident and significant, then, it will be necessary to reevaluate the recommendations of this geotechnical engineering report. It should be noted that it is the responsibility of you and your representative to notify CYG the variations immediately after the variation is observed.

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 submitted has been prepared in accordance with generally accepted geotechnical engineering practices. The conclusions and recommendations presented in this report are partly based on the evaluations of soils engineering information gathered from field exploration and laboratory tests, partly on experience, and partly on professional judgement. No warranty, either expressed or implied, is made or intended in connection with the above exploration or by the furnishing of this report or by any other oral or written statement.

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

APPENDIX A
FIELD EXPLORATION AND LABORATORY TESTING

1.0 FIELD EXPLORATION

Field exploration was performed by one of our engineers on January 9, 2020 and January 13, 2020 with the aid of a hollow-stem drill ring and hand laborers. Two (2) deep borings were explored to a depth of 51 feet at the locations as shown on Plate 1 for sampling of earth materials, liquefaction evaluations and foundation evaluations. Ten (10) shallow borings were explored to a maximum depth of 12 feet at the locations as shown on Plate 1 for sampling of earth materials and foundation evaluations. 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 soils were logged and then retained in plastic bags for laboratory particle size tests. The ring samples of onsite soils were retained in a series of brass rings, each having an inner diameter of 2.4 inches and a height of 1.0 inch. The soil samples and brass rings were then retained in plastic, close-fitting, moisture-tight containers. Bulk samples of onsite earth materials were collected for laboratory compaction test and expansion index test. The boring logs are presented in Plates A-1 to A-5.

2.0 LABORATORY TEST

Laboratory testing was performed after review of the 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 laboratory testing. The following engineering properties of onsite earth materials were determined by CYG: 1) field density and field moisture content, 2) maximum dry density and optimum moisture content, 3) cohesion and friction angle, 4) compressibility and hydro-consolidation, 5) expansion index, and 6) grain size distribution.

2.1 Moisture-Density Test

Onsite soils 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. Unit weights were determined in general accordance with ASTM Test Designation D2937. The results of moisture-density tests are summarized in Tables A.1 to A.3.

2.2 Direct Shear Tests

Three direct shear tests were performed 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 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, and 6) repeat step 5 to repeatedly reshear sample a minimum of 5 times or until a steady shear strength was recorded as a residual 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 in Plates DS-1 to DS-3.

2.3 Consolidation Tests

Fifteen consolidation tests were performed on selected ring samples to determine the compressibility and hydro-consolidation potential of alluvial soil. The consolidation tests were performed in general accordance with ASTM Standard D-2435. The ring samples were soil samples contained in a 2.4-inch-diameter and 1.0-inch-high sampling ring. This test was 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 pif) 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 the test are presented in terms of percent volume change versus applied vertical stress. The results of consolidation tests are presented in Plates CS-1 to CS-15.

2.4 Compaction Test

One compaction test was performed on one bulk soil sample to determine the maximum dry density and optimum moisture content of alluvial soil. The compaction test was performed in general accordance with ASTM Test Designation D1557. 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 compaction curves. The maximum dry density and optimum moisture content can be determined from the compaction curves. The results of compaction test are presented on Plate CM-1.

2.5 Expansion Index Test

One expansion index test was performed on one bulk soil sample to determine the expansion potential of alluvial soil. The expansion index test was performed in general accordance with expansion test procedures in ASTM D4829 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 48 and 52 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. Laboratory expansion index tests indicated an expansion index of 0 for the tested sample of alluvia soil.

2.6 Sieve Analysis and Hydrometer Test

Five (5) mechanic sieve tests and one (1) hydrometer test were performed on selected soil samples to determine their grain size distributions in accordance with ASTM Standard D-422-63(1998). 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 in Plates GS-1 to GS-5.

2.7 Chemical Test

Chemical tests were performed to determine the PH value, electrical resistivity, sulphate concentration and chloride concentration of onsite soils. The samples for chemical tests were retrieved by CYG. The chemical tests were performed by Project X Corrosion Engineering. The standards of CTM 643 were used in the determination of PH value and electrical resistivity. The standards of CTM 422 were used in the determination of chloride concentration. The standards of CTM 417 were used in the determination of sulfate concentration. The electrical resistivity test was performed under a saturated condition. The results of the chemical tests are presented in this appendix.

2.8 R value Test

The R value of onsite soil was tested. The sample for R-Value test was retrieved by CYG. The R-Value test was performed by pacific Materials Laboratory, Inc. The results of the R-Value test are presented in this appendix.

Table A.1. Results of Density and Moisture Tests for Samples from Boring B-1

Location	Depth ft	Soil Description	Dry Density pcf	Moisture Content, %	SPT N Value	Clay Content, %
B-1	2.5	Grayish brown sand (Qasa)	----	----	16	0
B-1	5	Brown sandy clayey silt (Qasa)	106	3	----	----
B-1	7.5	Grayish brown silty sand (Qasa)	----	----	17	0
B-1	10	Grayish brown silty sand (Qasa)	109	11	----	----
B-1	12.5	Grayish brown clayey silty sand (Qasa)	----	----	20	5
B-1	15	Grayish brown silty sand (Qasa)	111	16	----	----
B-1	17.5	Grayish brown gravelly sand (Qasa)	----	----	35	0
B-1	20	Grayish brown gravelly sand (Qasa)	109	4	----	----
B-1	22.5	Gray gravelly sand (Qasa)	----	----	35	0
B-1	25	Gray gravelly sand (Qasa)	116	2	----	----
B-1	27.5	Gray gravelly sand (Qasa)	----	----	43	0
B-1	30	Gray gravelly sand (Qasa)	122	2	----	----
B-1	32.5	Gray gravelly sand (Qasa)	----	----	43	0
B-1	35	Gray gravelly sand (Qasa)	118	6	----	----
B-1	37.5	Gray gravelly sand (Qasa)	----	----	> 100	0
B-1	40	Gray gravelly sand (Qasa)	121	5	----	----
B-1	42.5	Gray gravelly sand (Qasa)	----	----	47	0
B-1	45	Gray gravelly sand (Qasa)	118	2	----	----
B-1	47.5	Gray gravelly sand (Qasa)	----	----	> 100	0
B-1	50	Gray sandy gravel (Qasa)	129	2	----	----

Table A.2. Results of Density and Moisture Tests for Samples from Boring B-2

Location	Depth ft	Soil Description	Dry Density pcf	Moisture Content, %	SPT N Value	Clay Content, %
B-2	2.5	Light brown silty sand (Qasa)	----	----	12	0
B-2	5	Light brown silty sand (Qasa)	110	5	----	----
B-2	7.5	Light brown silty sand (Qasa)	----	----	16	0
B-2	10	Light brown clayey silty sand (Qasa)	110	18	----	----
B-2	12.5	Grayish brown silty sand (Qasa)	----	----	22	0
B-2	15	Grayish brown gravelly sand (Qasa)	111	1	----	----
B-2	17.5	Grayish brown gravelly sand (Qasa)	----	----	35	0
B-2	20	Grayish brown gravelly sand (Qasa)	136	4	----	----
B-2	22.5	Grayish brown clayey gravelly sand (Qasa)	----	----	32	< 5
B-2	25	Grayish brown clayey gravelly sand (Qasa)	117	6	----	----
B-2	27.5	Gray gravelly sand (Qasa)	----	----	37	0
B-2	30	Gray gravelly sand (Qasa)	126	5	----	----
B-2	32.5	Gray gravelly sand (Qasa)	----	----	> 100	0
B-2	35	Grayish brown clayey gravelly sand (Qasa)	117	5	----	----
B-2	37.5	Brown clayey silty sand (Qasa)	----	----	52	< 5
B-2	40	Gray gravelly sand (Qasa)	125	7	----	----
B-2	42.5	Gray gravelly sand (Qasa)	----	----	> 100	0
B-2	45	Gray gravelly sand (Qasa)	125	5	----	----
B-2	47.5	Gray gravelly sand (Qasa)	----	----	> 86	0
B-2	50	Gray gravelly sand (Qasa)	127	3	----	----

Table A.3. Results of Density and Moisture Tests for Samples from B-3 to B-12

Location	Depth ft	Soil Description	Dry Density pcf	Moisture Content %
B-3	1	Grayish brown sandy silt (Qasa)	115	7
B-3	4	Grayish brown brown sandy silt (Qasa)	109	2
B-3	7	Grayish brown silty sand with clay binder (Qasa)	100	11
B-3	10	Grayish brown silty sand with clay binder (Qasa)	93	20
B-4	2	Grayish brown clayey silty sand (Qasa)	113	3
B-4	6	Grayish brown sandy silt (Qasa)	104	3
B-4	10	Grayish brown sandy silt (Qasa)	104	3
B-5	1	Grayish brown silty sand (Qasa)	121	6
B-5	3	Grayish brown sandy silt (Qasa)	105	5
B-5	7	Grayish brown sandy silt (Qasa)	101	8
B-5	10	Grayish brown sandy silt (Qasa)	103	7
B-6	2	Grayish brown sandy silt (Qasa)	108	1
B-6	5	Brown clayey silty sand (Qasa)	107	18
B-6	9	Grayish brown silty sand (Qasa)	104	14
B-7	1	Brown silty sand (Qasa)	117	6
B-7	5	Grayish brown sand (Qasa)	106	2
B-7	12	Grayish brown silty sand (Qasa)	110	5
B-8	2	Brown silty sand (Qasa)	115	8
B-8	10	Grayish brown clayey silty sand (Qasa)	104	16
B-10	2	Grayish brown silty sand (Qasa)	112	2
B-10	6	Grayish brown silty sand (Qasa)	103	3
B-11	1	Light brown sand (Qasa)	105	3
B-11	6	Light brown silty sand (Qasa)	108	2
B-11	12	Grayish brown silty sand (Qasa)	103	3
B-12	2	Light brown silty sand (Qasa)	103	2
B-12	5	Light brown silty sand (Qasa)	104	4
B-12	8	Grayish brown silty sand (Qasa)	105	6
B-12	12	Grayish brown silty sand (Qasa)	105	1

BORING LOG

Exploration Date: January 9, 2020
Boring Method: Hollow Stem Drill Rig

Explored By: Paul Cai
Boring Diameter: 8 inches

Location	Depth ft	Blow Count (N)	Soils Descriptions
B - 1			**Alluvium (0'- 51')**
	0 - 2.5	--	Light brown silty sand, dry to slightly moist, loose to moderately dense
	2.5 - 5	16 @ 2.5'	Grayish brown sand, slightly moist, moderately dense
	5 - 7.5		Grayish brown silty sand, slightly moist, moderately dense
	7.5 - 10	17 @ 7.5'	Grayish brown clayey silty sand, moist, moderately dense
	10 - 12.5		Grayish brown silty sand, moist, moderately dense
	12.5 - 15	20 @ 12.5'	Grayish brown silty sand, moist, moderately dense
	15 - 17.5		Grayish brown silty sand, moist, moderately dense
	17.5 - 20	35 @17.5'	Grayish brown gravelly sand, moist, moderately dense
	20 - 22.5		Grayish brown gravelly sand, moist, moderately dense
	22.5 - 25	35 @ 22.5'	Gray gravelly sand, moist, moderately dense
	25 - 27.5		Gray gravelly sand, moist, moderately dense
	27.5 - 30	43 @ 27.5'	Gray gravelly sand, moist, moderately dense
	30 - 32.5		Gray gravelly sand, moist, moderately dense
	32.5 - 35	43 @ 32.5'	Gray gravelly sand, moist, moderately dense
	35 - 37.5		Gray gravelly sand, moist, moderately dense
	37.5 - 40	> 100 @ 37.5'	Gray gravelly sand, moist, moderately dense
	40 - 42.5		Gray gravelly sand, moist, moderately dense
	42.5 - 45	47 @ 42.5'	Gray gravelly sand, moist, moderately dense
	45 - 47.5		Gray gravelly sand, moist, moderately dense
	47.5 - 50	>100 @ 47.5'	Gray gravelly sand, moist, moderately dense
	50 - 51		Gray gravelly sand, moist, moderately dense
			Ends at 51 ft. No water. No caving. Ring samples at: 5, 10, 15, 20, 25, 30, 35, 40, 45, 50 ft SPT samples at 7.5, 12.5, 17.5, 22.5, 27.5, 32.5, 37.5, 42.5, 47.5 ft.

Plate A-1

BORING LOG

Exploration Date: January 9, 2020
Boring Method: Hollow Stem Drill Rig

Explored By: Paul Cai
Boring Diameter: 8 inches

Location	Depth ft	Blow Count (N)	Soils Descriptions
B - 2			**Alluvium (0'- 51')**
	0 - 2.5	--	Light brown silty sand, dry to slightly moist, moderately dense
	2.5 - 5	12 @ 2.5'	Light brown sand, slightly moist, moderately dense
	5 - 7.5		Light brown silty sand, slightly moist, moderately dense
	7.5 - 10	16 @ 7.5'	Light brown silty sand, moist, moderately dense
	10 - 12.5		Light brown silty sand, moist, moderately dense
	12.5 - 15	22 @ 12.5'	Grayish brown silty sand, moist, moderately dense
	15 - 17.5		Grayish brown gravelly sand, moist, moderately dense
	17.5 - 20	35 @17.5'	Grayish brown gravelly sand, moist, moderately dense
	20 - 22.5		Grayish brown gravelly sand, moist, moderately dense
	22.5 - 25	32 @ 22.5'	Gray gravelly sand with clay binder, moist, moderately dense
	25 - 27.5		Gray gravelly sand with clay binder, moist, moderately dense
	27.5 - 30	37 @ 27.5'	Gray gravelly sand, moist, moderately dense
	30 - 32.5		Gray gravelly sand, moist, moderately dense
	32.5 - 35	> 100 @ 32.5'	Gray gravelly sand, moist, moderately dense
	35 - 37.5		Gray gravelly sand with clay binder , moist, moderately dense
	37.5 - 40	52 @ 37.5'	Brown clayey silty sand, moist, moderately dense
	40 - 42.5		Gray gravelly sand, moist, moderately dense
	42.5 - 45	> 100 @ 42.5'	Gray gravelly sand, moist, moderately dense
	45 - 47.5		Gray gravelly sand, moist, moderately dense
	47.5 - 50	> 86 @ 47.5'	Gray gravelly sand, moist, moderately dense
	50 - 51		Gray gravelly sand, moist, moderately dense
			Ends at 51 ft. No water. No caving. Ring samples at: 5, 10, 15, 20, 25, 30, 35, 40, 45, 50 ft SPT samples at 7.5, 12.5, 17.5, 22.5, 27.5, 32.5, 37.5, 42.5, 47.5 ft.

Plate A-2

BORING LOG

Exploration Date: January 9, 2020
Boring Method: Hollow Stem Drill Rig

Explored By: Paul Cai
Boring Diameter: 8 inches

Location	Depth ft	Soils Descriptions
B-3		
	0 - 0.5	**Artificial Fill (0 - 0.5')** Light brown silty sand, dry, loose
	0.5 - 4	**Alluvium (0.5 - 10')** Grayish brown silty sand, slightly moist, moderately dense
	4 - 7	Brown silty sand, slightly moist, moderately dense
	7 - 10	Grayish brown silty sand with clay binder, slightly moist, moderately dense
		Ends at 10 ft. No water. No caving. Samples at 1, 4, 7, 10 ft.
B-4		
	0 - 0.5	**Artificial Fill (0 - 0.5')** Light brown silty sand, dry, loose
	0.5 - 10	**Alluvium (0.5 - 10')** Grayish brown silty sand, slightly moist, moderately dense
		Ends at 10 ft. No water. No caving. Samples at 2, 6, 10 ft.
B-5		
	0 - 1	**Artificial Fill (0 - 1')** Light brown silty sand, dry, loose
	1 - 10	**Alluvium (0.5 - 10')** Grayish brown silty sand, slightly moist, moderately dense
		Ends at 10 ft. No water. No caving. Samples at 1, 3, 7, 10 ft.

Plate A-3

BORING LOG

Exploration Date: January 9, 2020 and January 13, 2020
Boring Method: Hollow Stem Drill Rig & Hand Auger

Explored By: Paul Cai
Boring Diameter: 4 & 8 inches

Location	Depth ft	Soils Descriptions
B-6		
	0 - 2	**Artificial Fill (0 - 2')** Light brown silty sand, dry, loose
	2 - 4	**Alluvium (2' - 11')** Grayish brown silty sand, slightly moist, moderately dense
	4 - 8	Brown clayey silty sand, slightly moist, moderately dense
	8 - 11	Grayish brown silty sand, moist, moderately dense
		Ends at 11 ft. No water. No caving. Samples at 2, 5, 9 ft.
B-7	0 - 1	**Artificial Fill (0 - 1')** Light brown silty sand, dry, loose
	1 - 12	**Alluvium (1' - 12')** Grayish brown silty sand, slightly moist, moderately dense
		Ends at 12 ft. No water. No caving. Samples at 1, 5, 12 ft.
B-8		
	0 - 2	**Artificial Fill (0 - 2')** Light brown silty sand, dry, loose
	2 - 10	**Alluvium (2' - 12')** Grayish brown silty sand, slightly moist, moderately dense
		Ends at 12 ft. No water. No caving. Samples at 2, 10 ft.
B-9		
	0 - 1	**Artificial Fill (0 - 2')** Light brown silty sand, dry, loose
	1 - 10	**Alluvium (2' - 12')** Grayish brown silty sand, slightly moist, moderately dense
		Ends at 12 ft. No water. Caving. No sample was taken.

Plate A-4

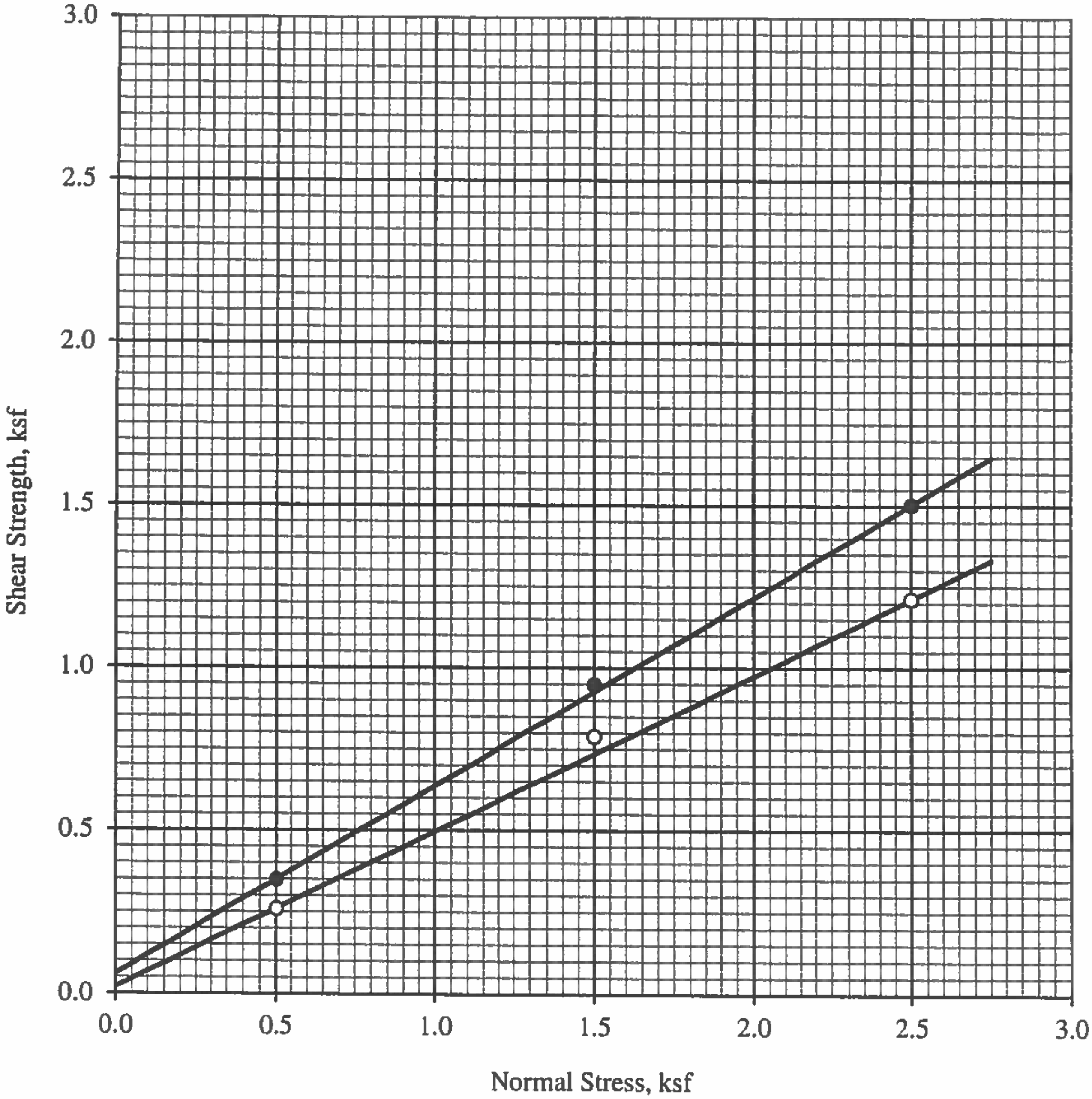
BORING LOG

Exploration Date: January 9, 2020
Boring Method: Hollow Stem Drill Rig

Explored By: Paul Cai
Boring Diameter: 8 inches

Location	Depth ft	Soils Descriptions
B-10	0 - 2	**Artificial Fill (0 - 2')** Light brown silty sand, dry, loose
	2 - 12	**Alluvium (2' - 11')** Grayish brown silty sand, slightly moist, moderately dense
		Ends at 12 ft. No water. No caving. Samples at 2, 6 ft.
B-11	0 - 1	**Artificial Fill (0 - 1')** Light brown silty sand, dry, loose
	1 - 12	**Alluvium (1' - 12')** Light brown and grayish brown silty sand, slightly moist, moderately dense
		Ends at 12 ft. No water. No caving. Samples at 1, 6, 12 ft.
B-12	0 - 2	**Artificial Fill (0 - 2')** Light brown silty sand, dry, loose
	2 - 5	**Alluvium (2' - 12')** Light brown sand, slightly moist, moderately dense
	5 - 12	Grayish brown silty sand, slightly moist, moderately dense
		Ends at 12 ft. No water. No caving. Samples at 2, 5, 8, 12 ft.

Plate A-5



● Peak	- At Saturation Moisture Content	C =	60 psf	ϕ = 30 °
○ Residual	- At Saturation Moisture Content	C =	20 psf	ϕ = 25.5 °

Field Dry Density = 101 pcf
Field Moisture Content = 3 %
Saturation Moisture Content = 25 %

Boring : B-1
Depth : 5 ft
Description : Grayish silty sand

C. Y. GEOTECH, INC.

Geotechnical Engineering
and Engineering Geology

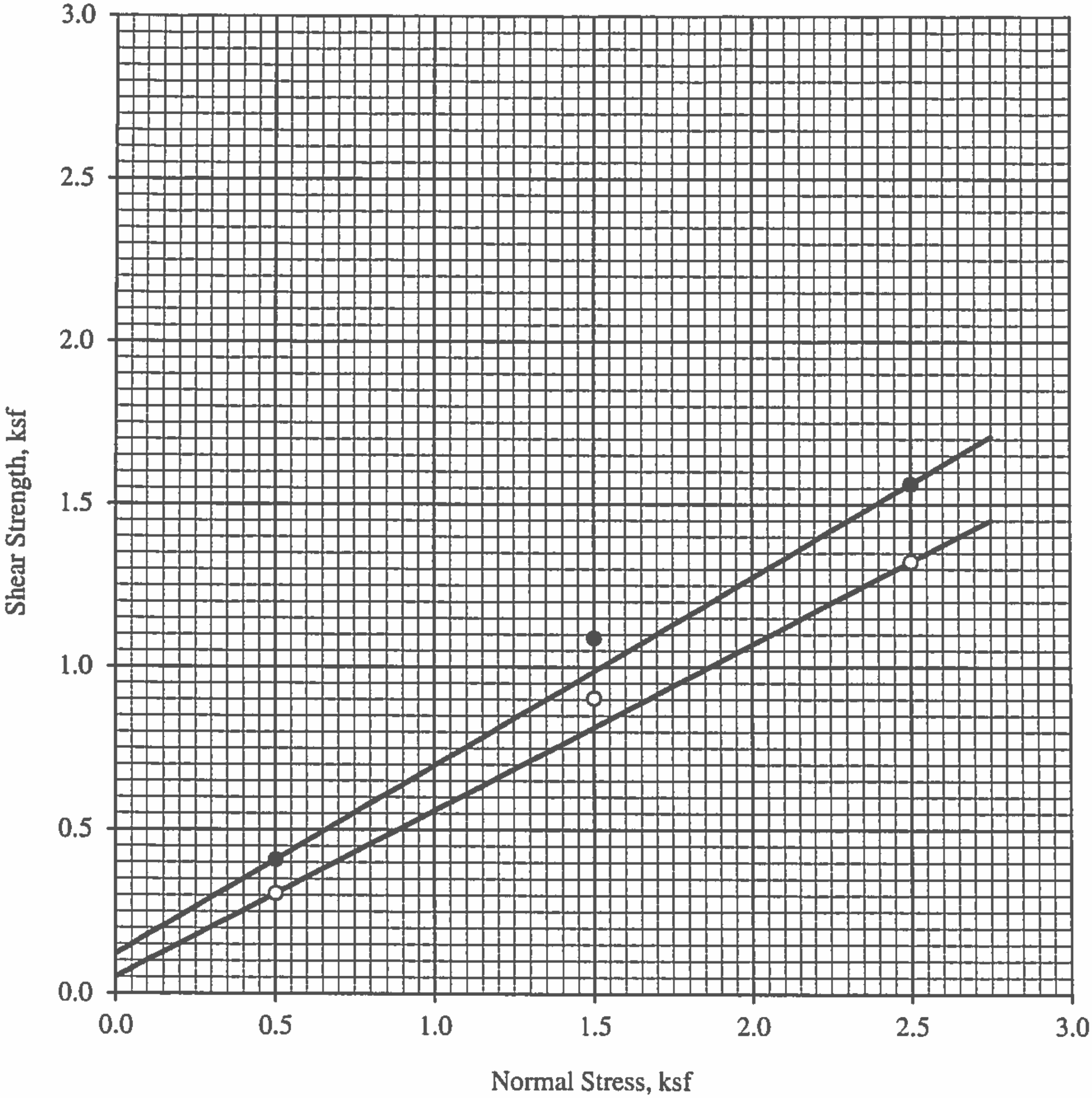
Giant Inland Empire RV Center Inc.

Date : 01-2020

P.N. No.: CYG-19-8863

Shear Diagram

Plate DS - 1



● Peak	- At Saturation Moisture Content	C = 120 psf	ϕ = 30 °
○ Residual	- At Saturation Moisture Content	C = 50 psf	ϕ = 27 °

Field Dry Density = 117 pcf
Field Moisture Content = 6 %
Saturation Moisture Content = 16 %

Boring : B-7
Depth : 1 ft
Description : Brown silty sand

C. Y. GEOTECH, INC.

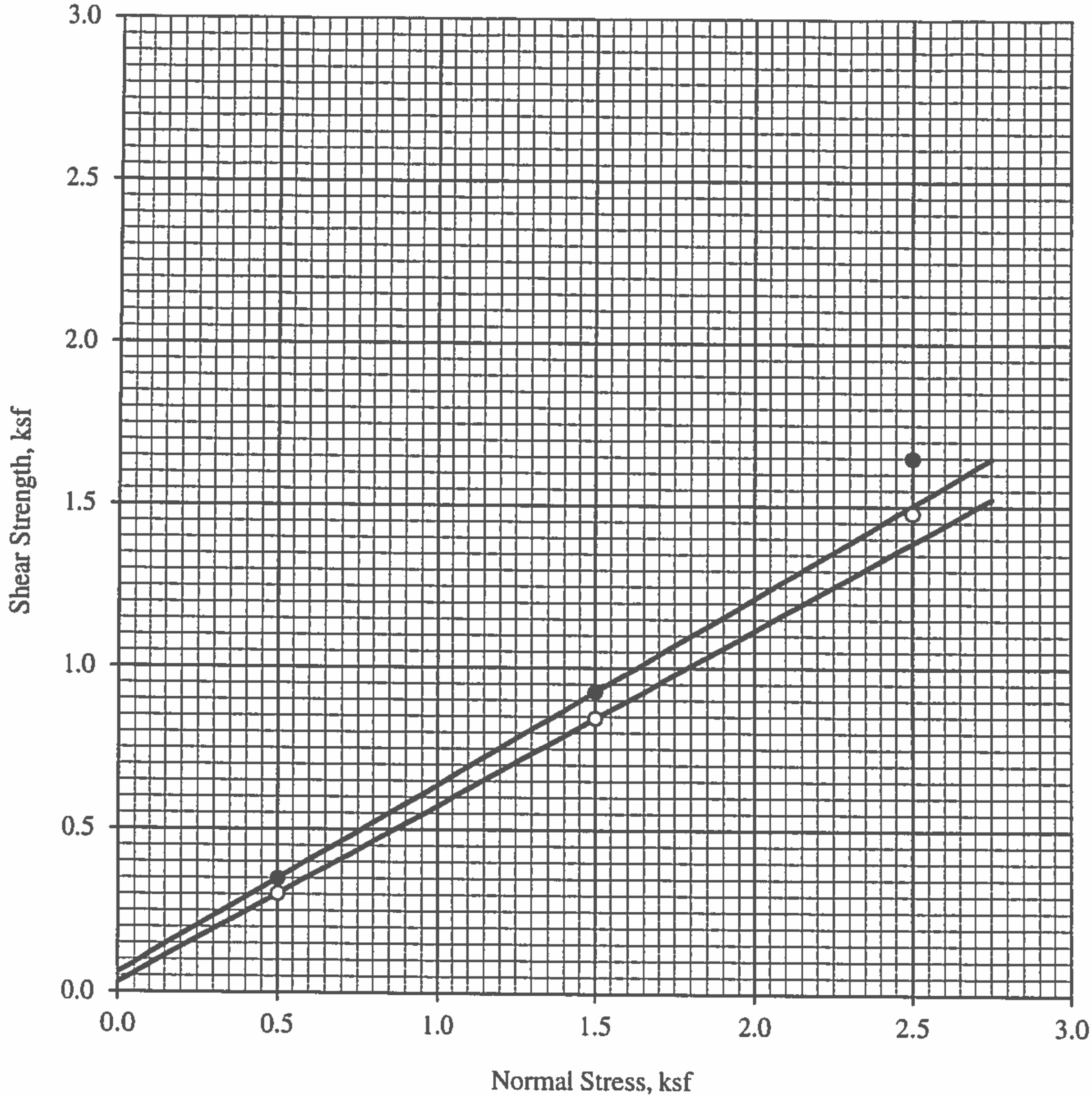
Geotechnical Engineering
and Engineering Geology

Giant Inland Empire RV Center Inc.

Date : 01-2020 | P.N. No.: CYG-19-8863

Shear Diagram

Plate DS - 2



● Peak - At Saturation Moisture Content C = 60 psf ϕ = 30 °

○ Residual - At Saturation Moisture Content C = 30 psf ϕ = 28.5 °

Field Dry Density = 98 pcf Field Moisture Content = 2 % Saturation Moisture Content = 27 %	Boring : B-12 Depth : 2 ft Description : Light brown silty sand

C. Y. GEOTECH, INC.

Geotechnical Engineering
and Engineering Geology

Giant Inland Empire RV Center Inc.

Date : 01-2020

P.N. No.: CYG-19-8863

Shear Diagram

Plate DS - 3

C. Y. GEOTECH, INC.

Geotechnical Engineering and Engineering Geology

Giant Inland Empire RV Center Inc.

Date : 01-2020 | P.N. No: CYG-19-8863

Consolidation Test

Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
B-1	5	3	21	1.0	2.4

Classification : Grayish brown silty sand

Hydroconsolidation = 0.9 %

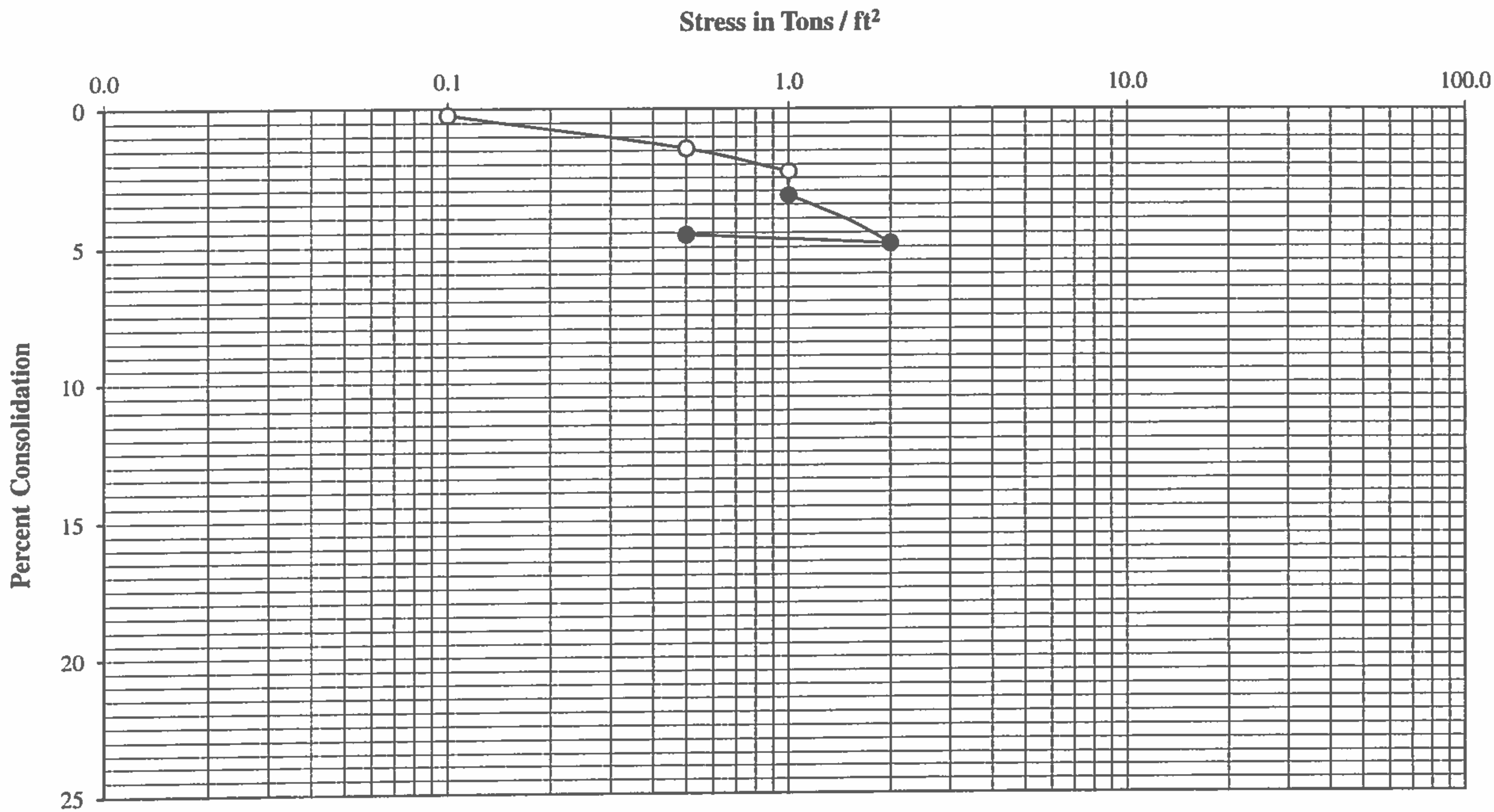


Plate CS - 1

C. Y. GEOTECH, INC.

Geotechnical Engineering and Engineering Geology

Giant Inland Empire RV Center Inc.

Date : 01-2020 | P.N. No: CYG-19-8863

Consolidation Test

Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
B-1	10	11	20	1.0	2.4

Classification : Grayish brown clayey silty sand

Hydroconsolidation = 0.2 %

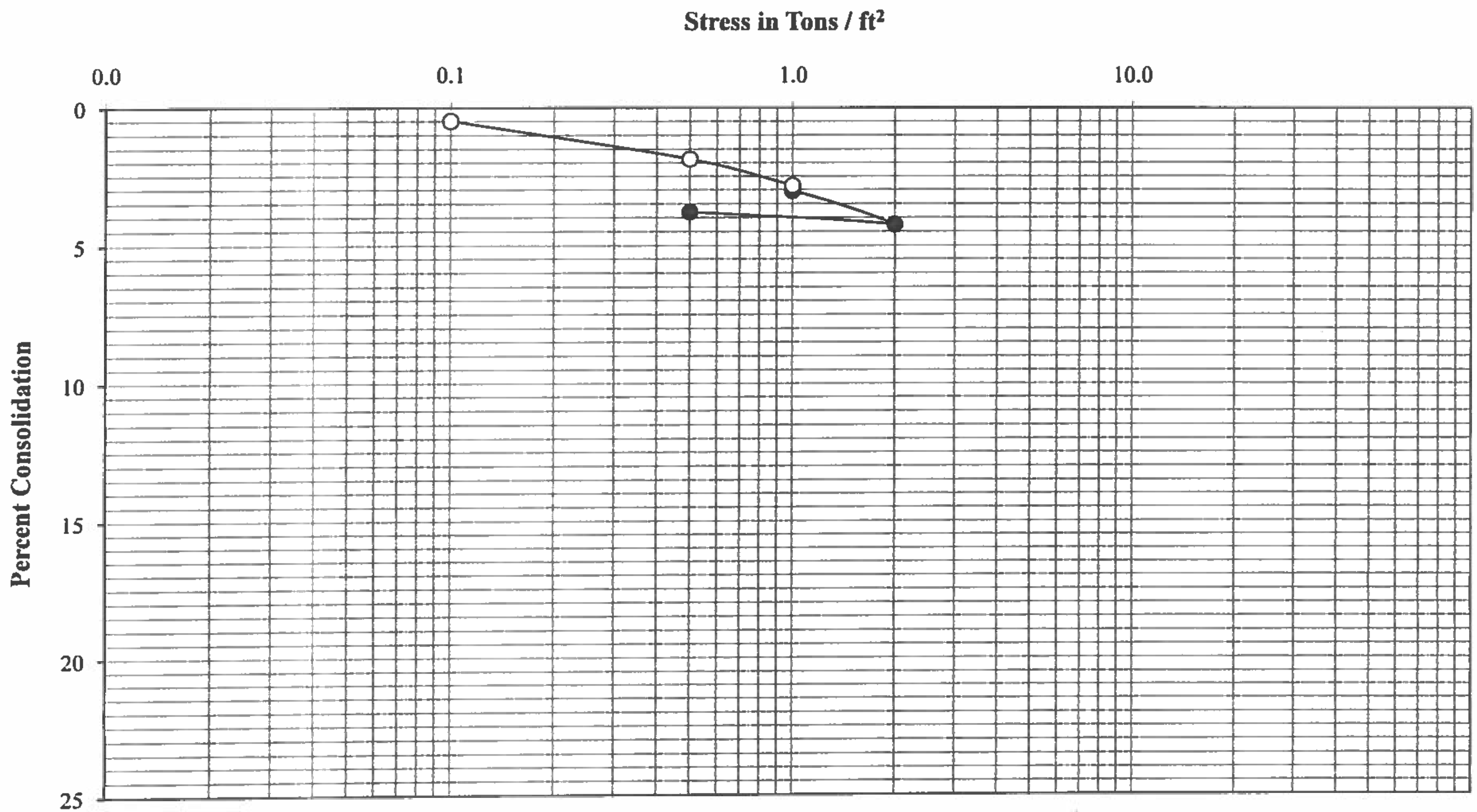


Plate CS - 2

C. Y. GEOTECH, INC.

Geotechnical Engineering and Engineering Geology

Giant Inland Empire RV Center Inc.

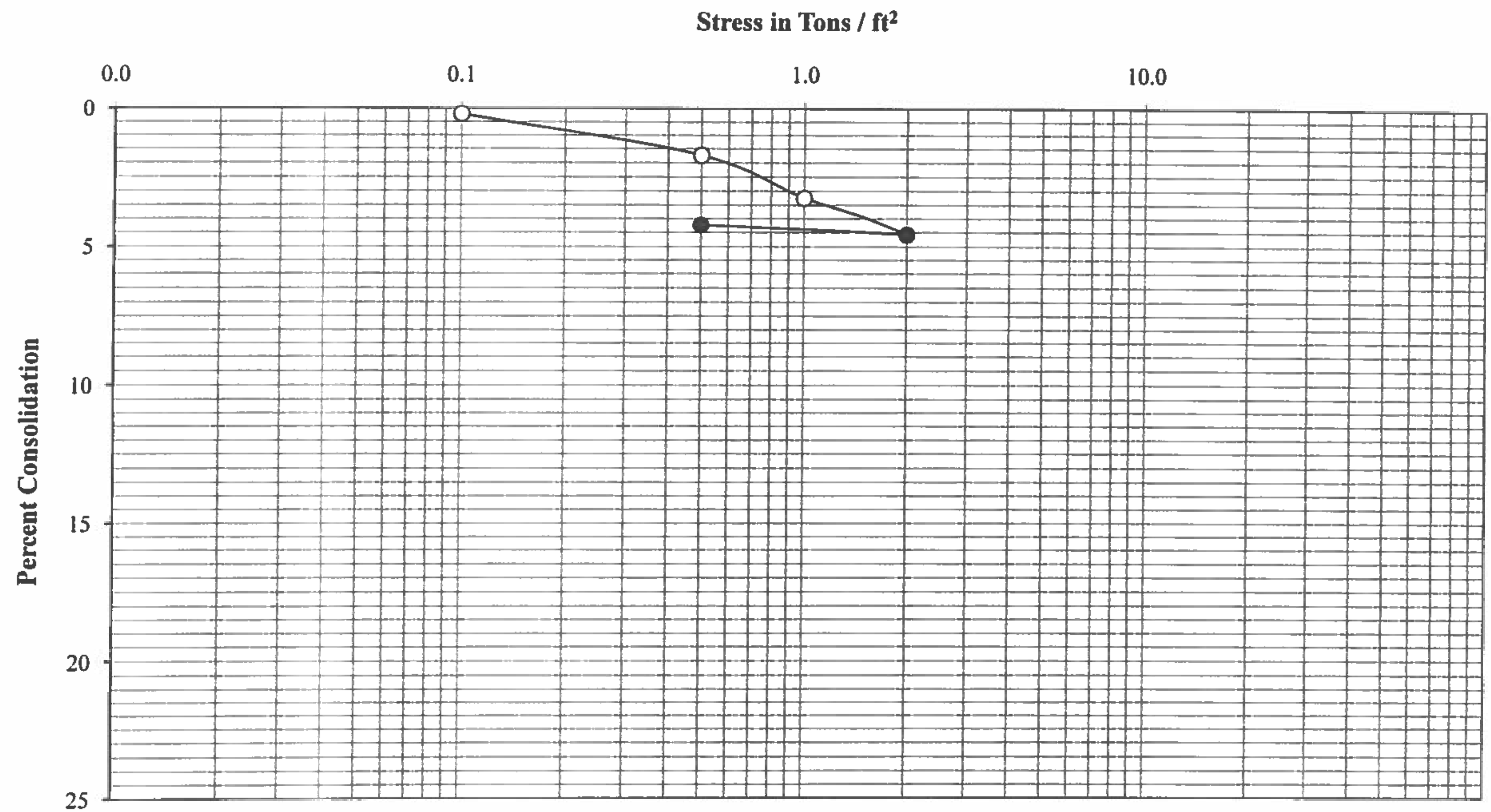
Date : 01-2020 | P.N. No: CYG-19-8863

Consolidation Test

Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
B-1	15	16	21	1.0	2.4

Classification : Grayish brown silty sand

Hydroconsolidation = 0 %



C. Y. GEOTECH, INC.

Geotechnical Engineering and Engineering Geology

Giant Inland Empire RV Center Inc.

Date : 01-2020 | P.N. No: CYG-19-8863

Consolidation Test

Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
B-2	5	6	21	1.0	2.4

Classification : Light brown silty sand

Hydroconsolidation = 0.7 %

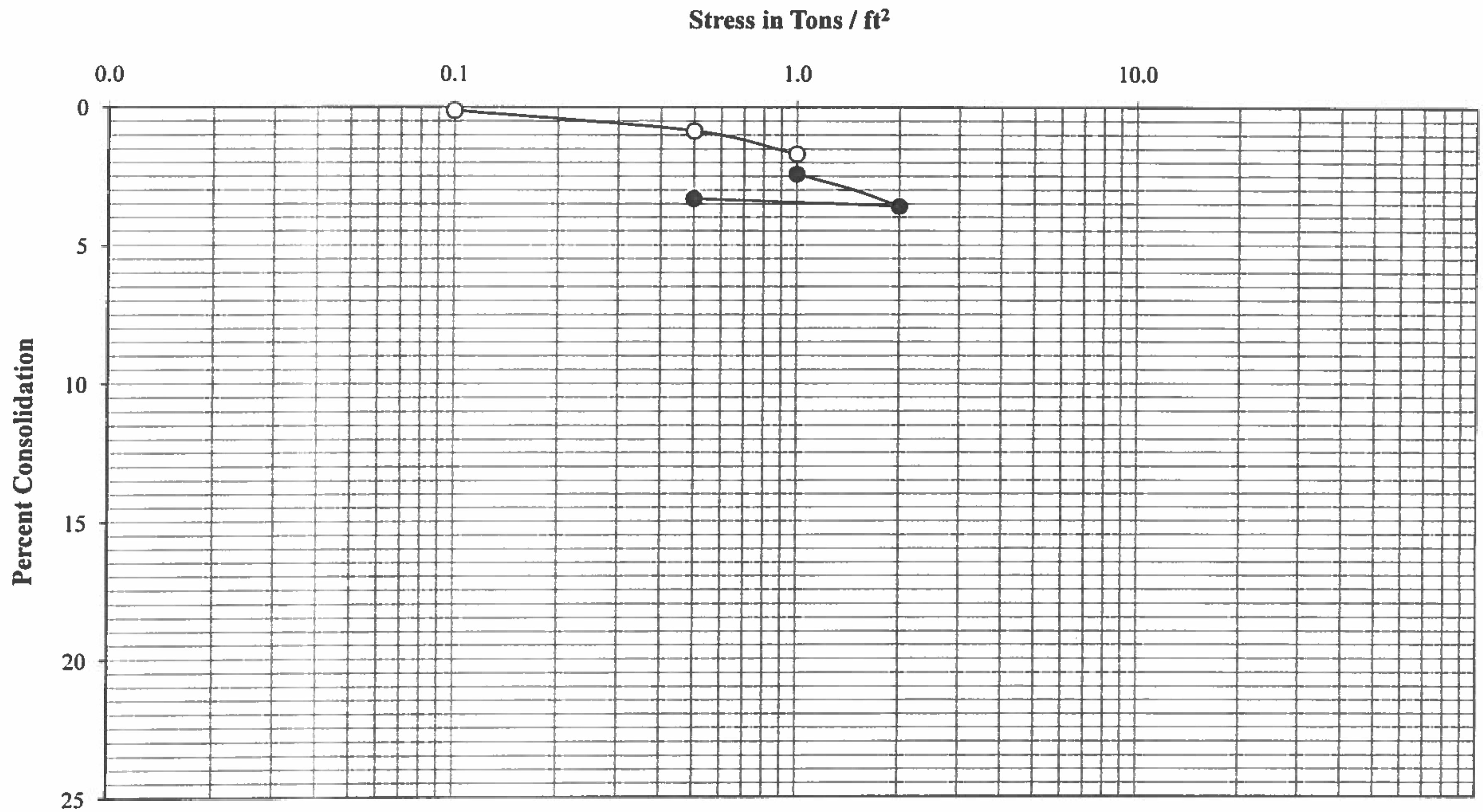


Plate CS - 4

C. Y. GEOTECH, INC.

Geotechnical Engineering and Engineering Geology

Giant Inland Empire RV Center Inc.

Date : 01-2020	P.N. No: CYG-19-8863

Consolidation Test

Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
B-2	10	18	19	1.0	2.4

Classification : Light brown silty sand

Hydroconsolidation = 0.9 %

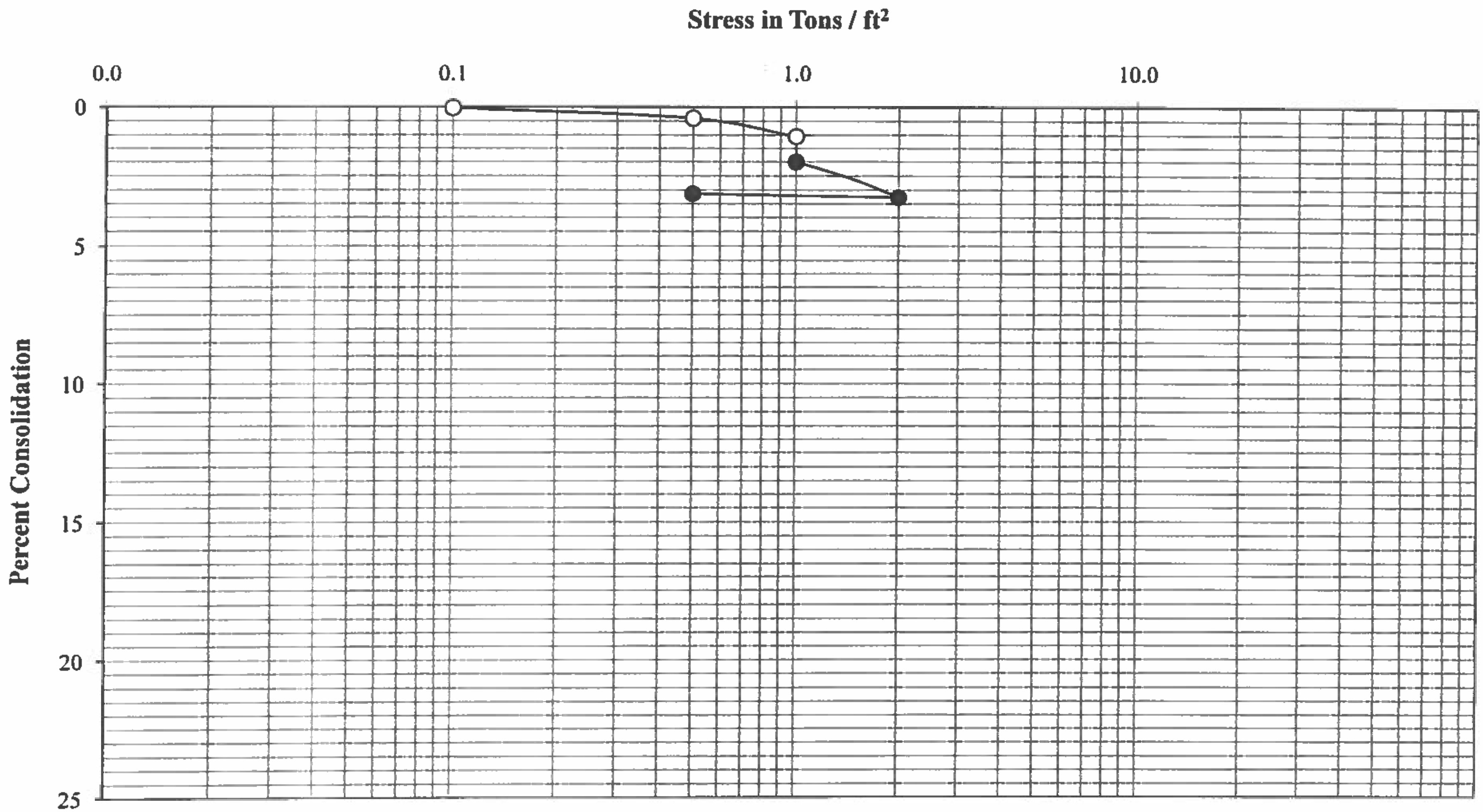


Plate CS - 5

C. Y. GEOTECH, INC.

Geotechnical Engineering and Engineering Geology

Giant Inland Empire RV Center Inc.

Date : 01-2020	P.N. No: CYG-19-8863

Consolidation Test

Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
B-3	4	2	20	1.0	2.4

Classification : Grayish brown sandy silt

Hydroconsolidation = 0.4 %

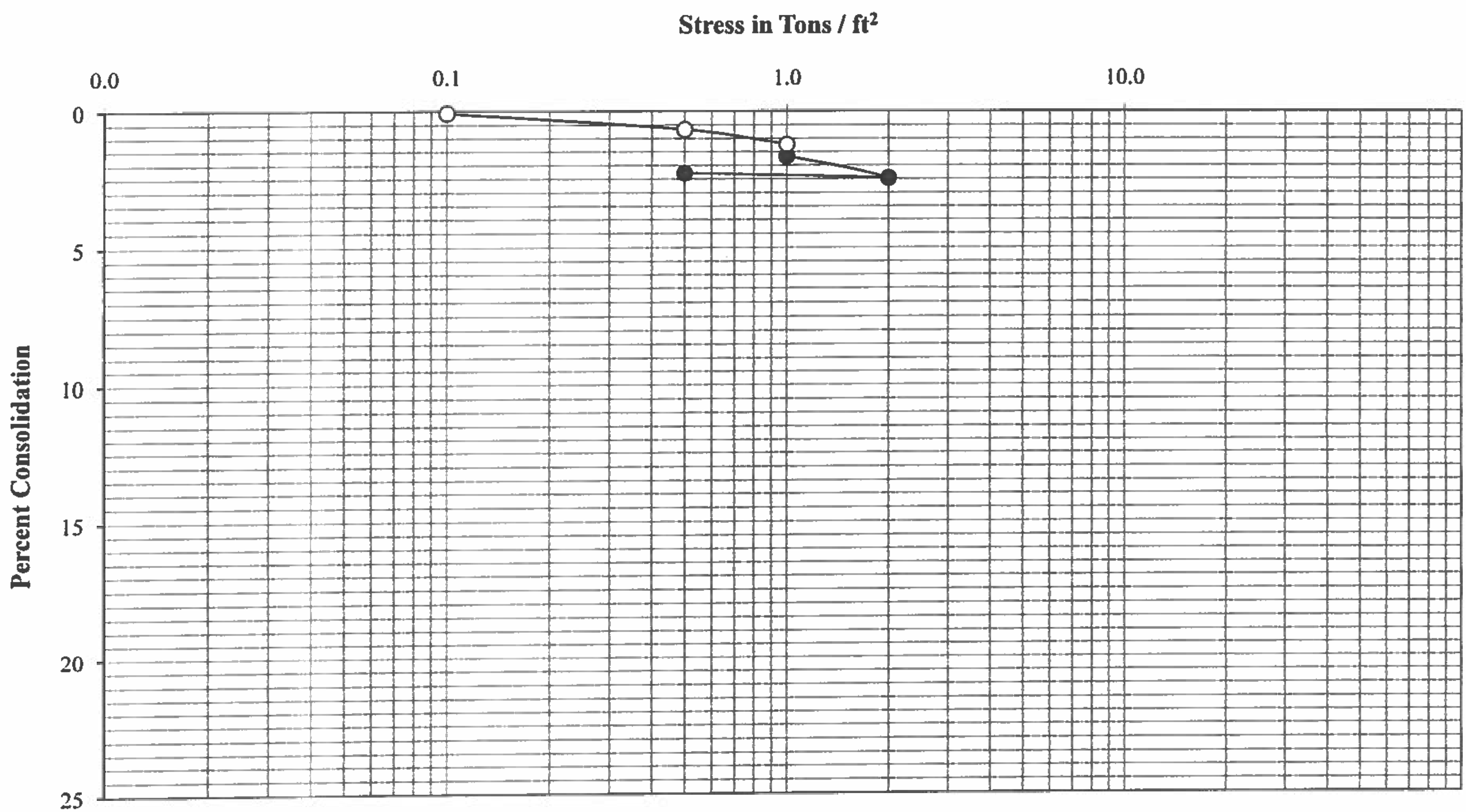


Plate CS - 6

C. Y. GEOTECH, INC.

Geotechnical Engineering and Engineering Geology

Giant Inland Empire RV Center Inc.

Date : 01-2020

P.N. No: CYG-19-8863

Consolidation Test

Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
B-3	7	11	25	1.0	2.4

Classification : Grayish brown silty sand

Hydroconsolidation = 0.1 %

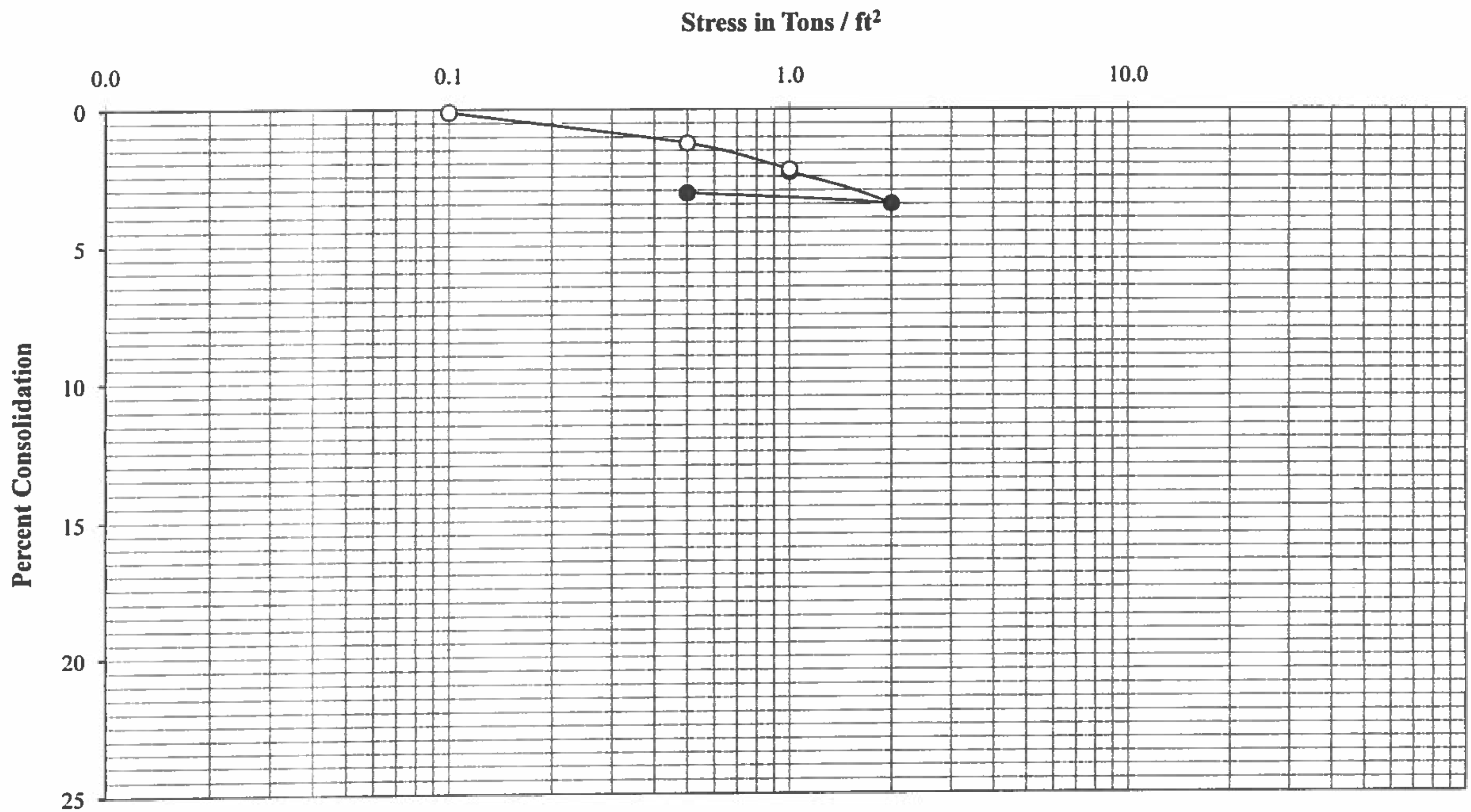


Plate CS - 7

C. Y. GEOTECH, INC.

Geotechnical Engineering and Engineering Geology

Giant Inland Empire RV Center Inc.

Date : 01-2020	P.N. No: CYG-19-8863

Consolidation Test

Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
B-7	5	2	21	1.0	2.4

Classification : Grayish brown sand

Hydroconsolidation = 0.3 %

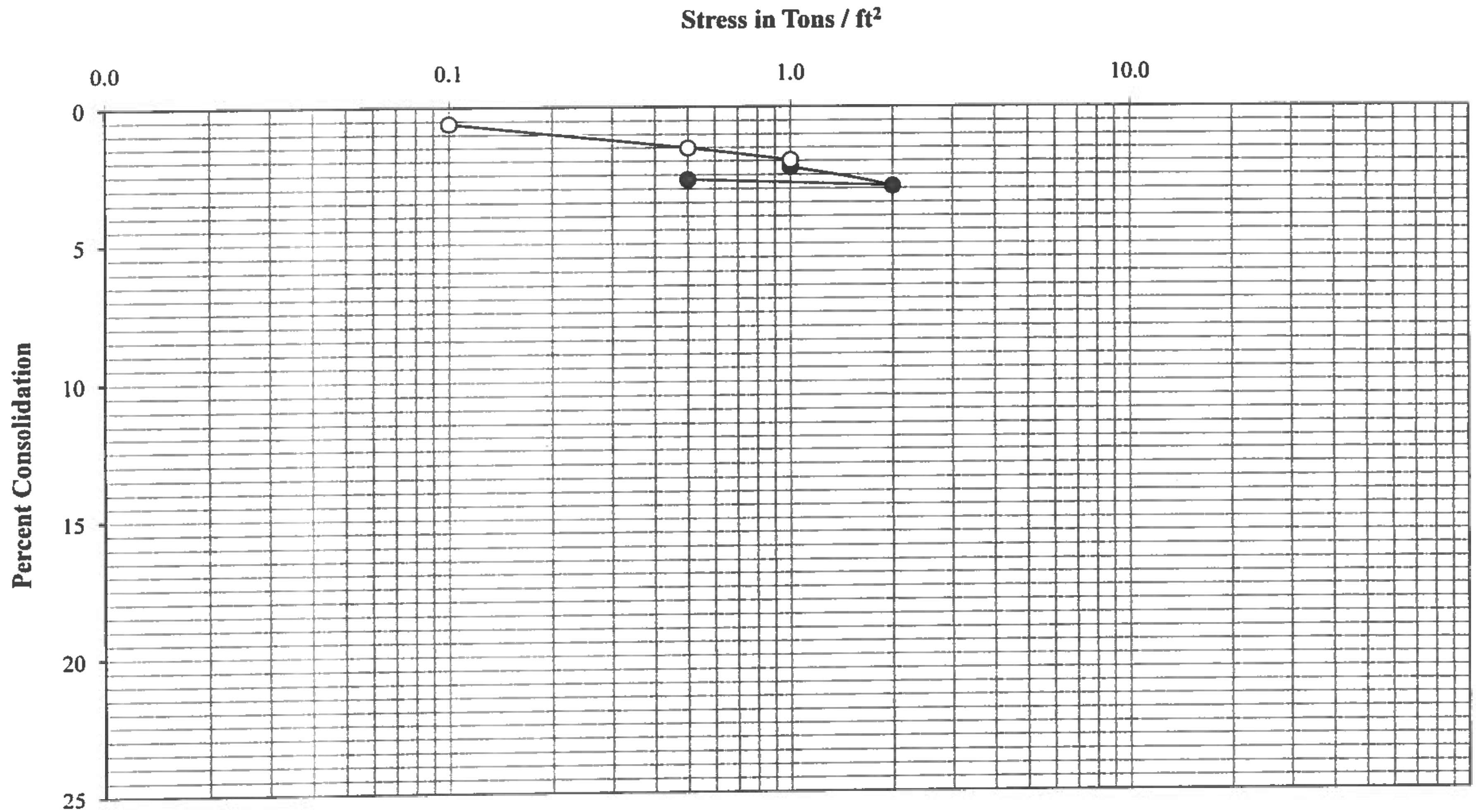


Plate CS - 8

C. Y. GEOTECH, INC.

Geotechnical Engineering and Engineering Geology

Giant Inland Empire RV Center Inc.

Date : 01-2020 | P.N. No: CYG-19-8863

Consolidation Test

Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
B-8	2	8	17	1.0	2.4

Classification : Brown silty sand

Hydroconsolidation = 0.6 %

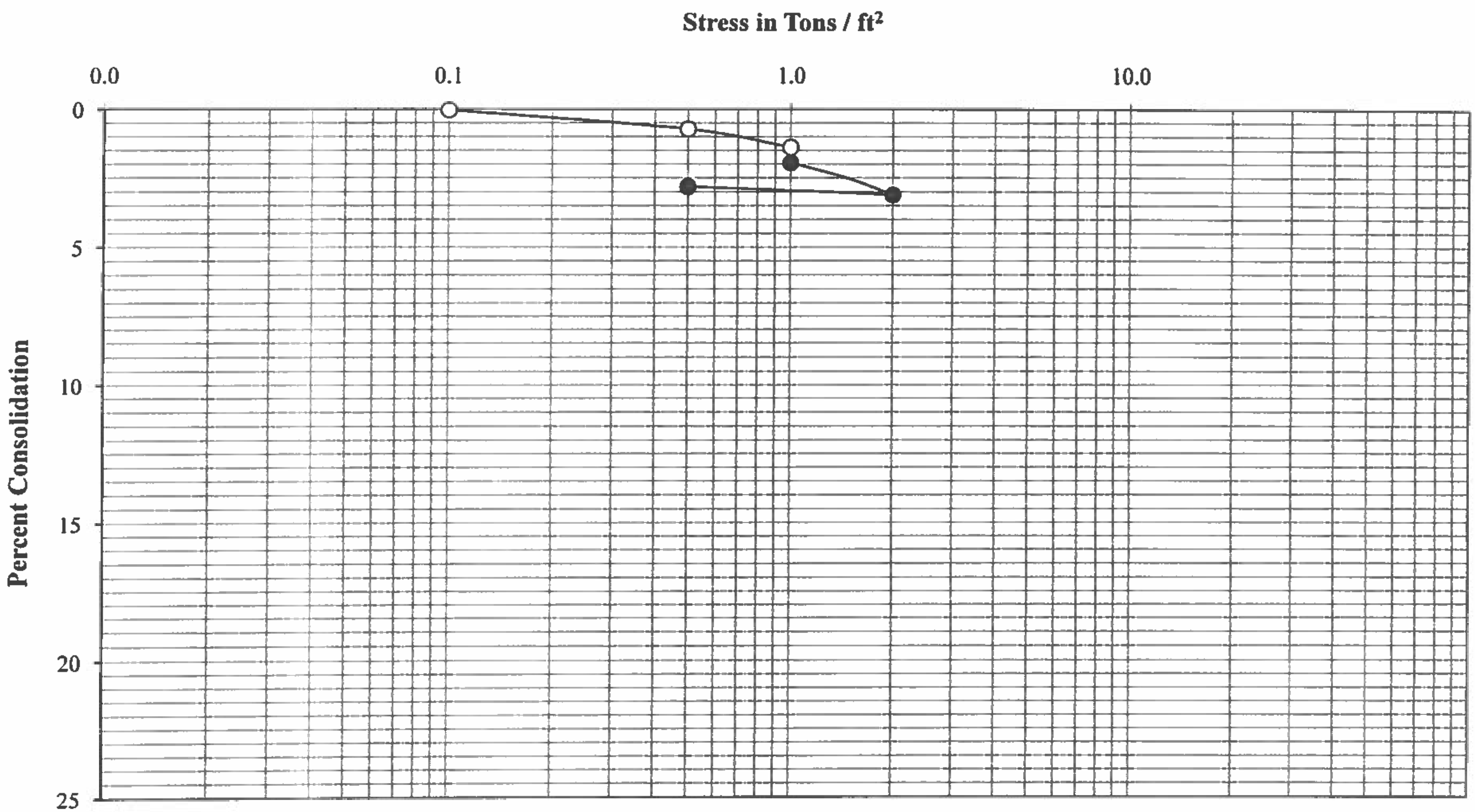


Plate CS - 9

C. Y. GEOTECH, INC.

Geotechnical Engineering and Engineering Geology

Giant Inland Empire RV Center Inc.

Date : 01-2020

P.N. No: CYG-19-8863

Consolidation Test

Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
B-12	2	2	22	1.0	2.4

Classification : Light brown silty sand

Hydroconsolidation = 0.6 %

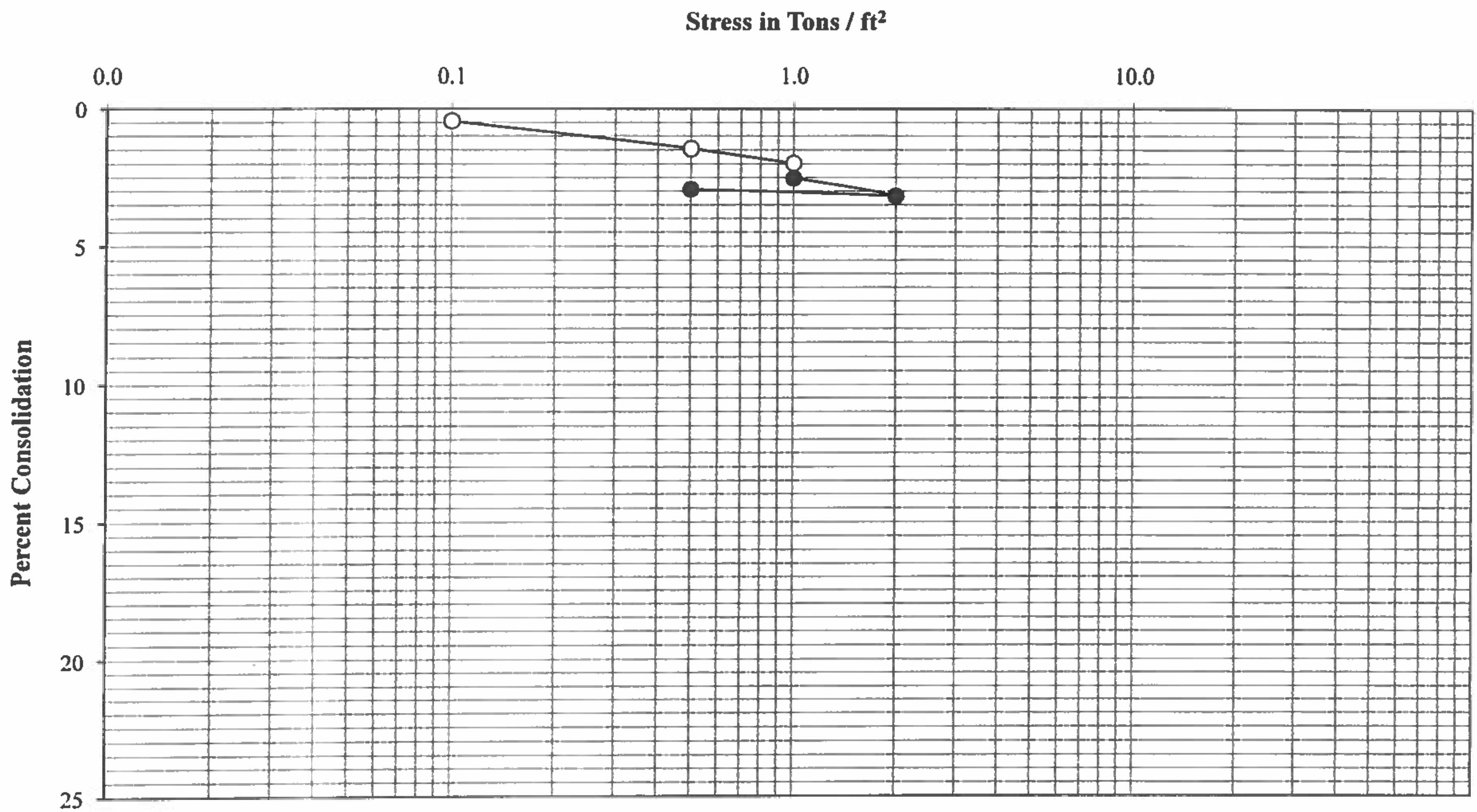


Plate CS - 10

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

Giant Inland Empire RV Center Inc.

Date : 01-2020 | P.N. No: CYG-19-8863

Consolidation Test

Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
B-12	5	4	23	1.0	2.4

Classification : Light brown silty sand
Hydroconsolidation = 0.9 %

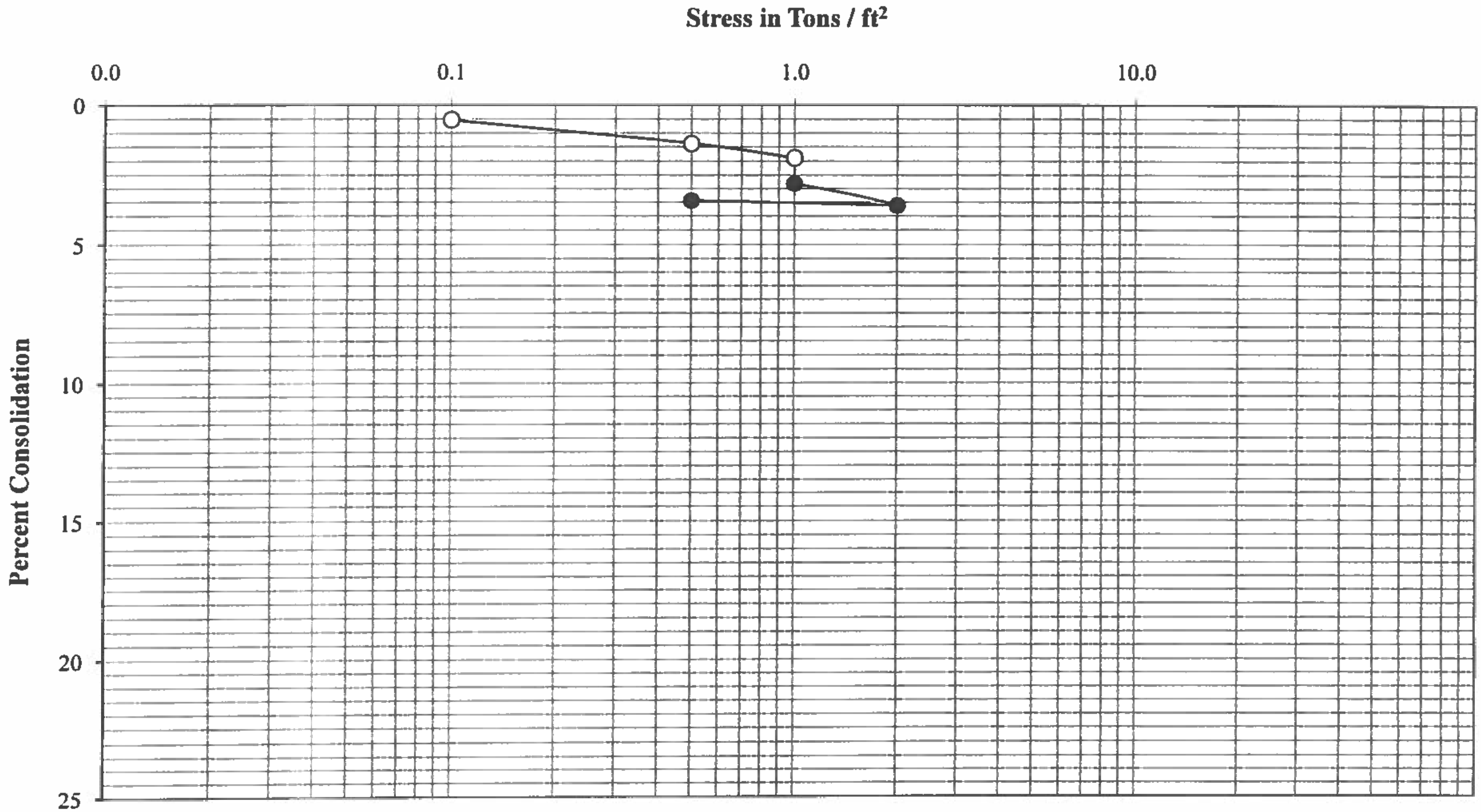


Plate CS - 11

C. Y. GEOTECH, INC.

Geotechnical Engineering and Engineering Geology

Giant Inland Empire RV Center Inc.

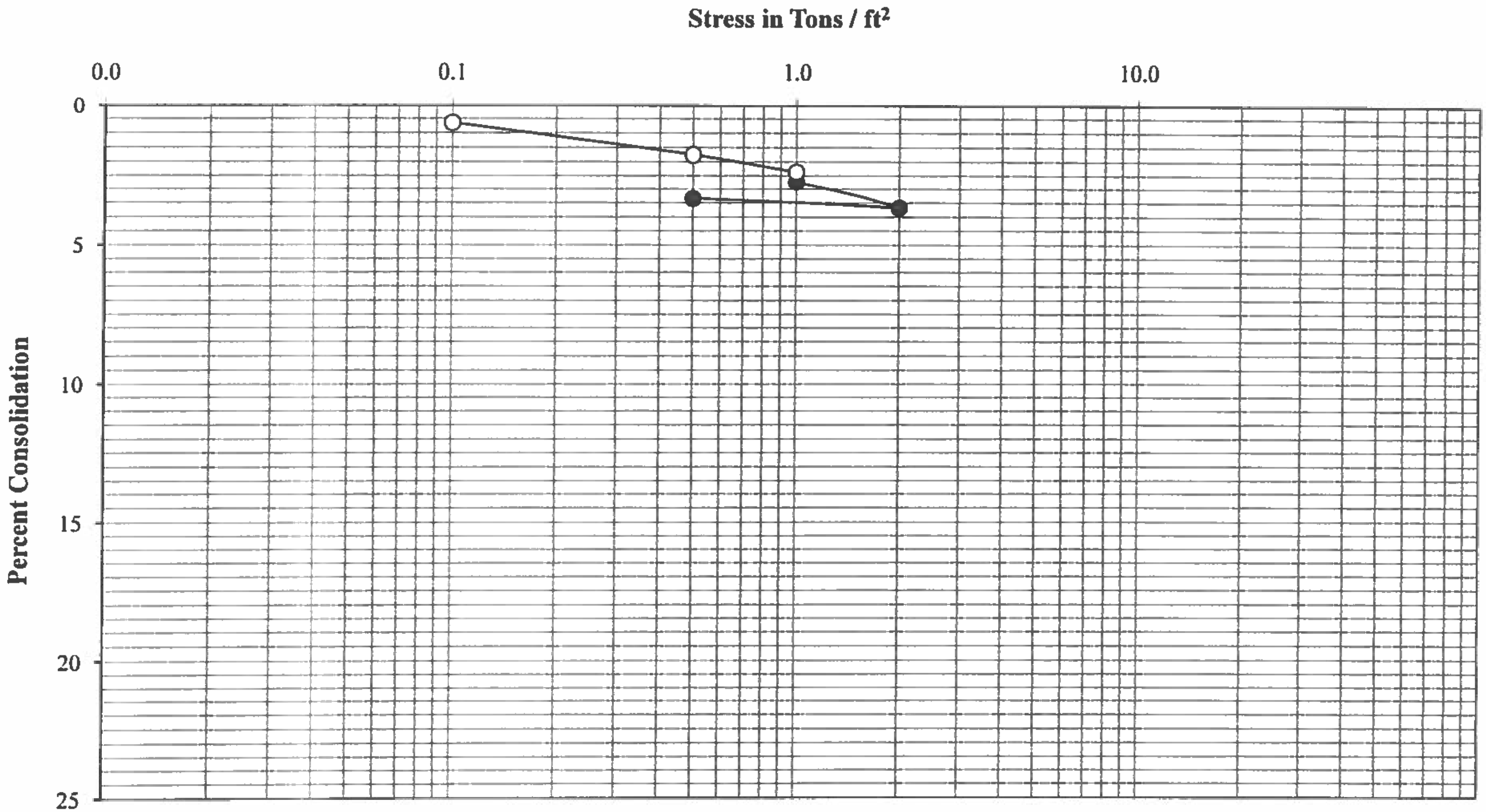
Date : 01-2020 | P.N. No: CYG-19-8863

Consolidation Test

Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
B-12	8	6	22	1.0	2.4

Classification : Grayish brown silty sand

Hydroconsolidation = 0.4 %



C. Y. GEOTECH, INC.

Geotechnical Engineering and Engineering Geology

Giant Inland Empire RV Center Inc.

Date : 01-2020

P.N. No: CYG-19-8863

Consolidation Test

Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
B-12	12	1	22	1.0	2.4

Classification : Grayish brown sand

Hydroconsolidation = 0.2 %

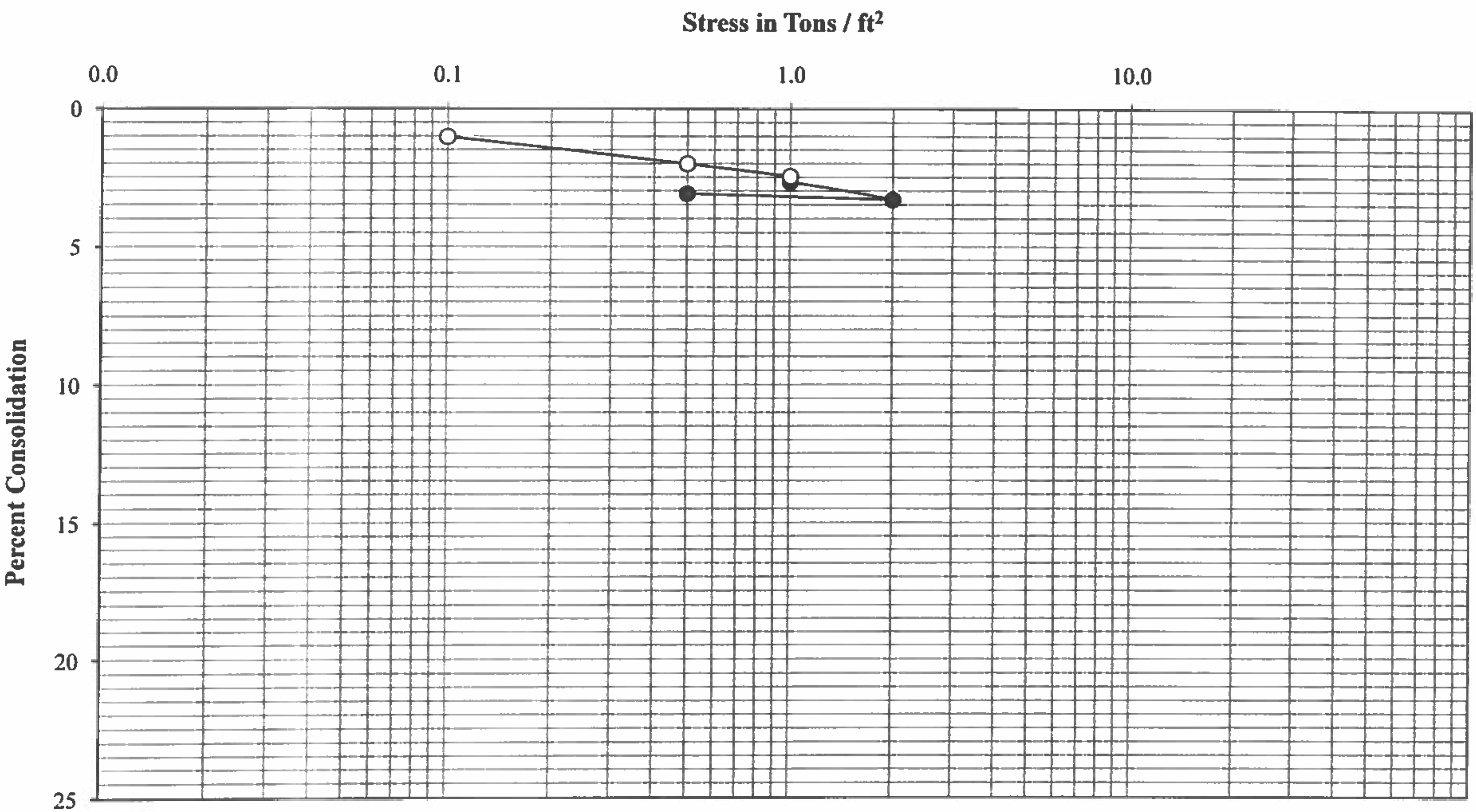


Plate CS - 13

C. Y. GEOTECH, INC.

Geotechnical Engineering and Engineering Geology

Giant Inland Empire RV Center Inc.

Date : 01-2020 | P.N. No: CYG-19-8863

Consolidation Test

Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
Onsite Soil	-- --	7.5	15	1.0	2.4

Classification : Brown clayey sand silt (Remolded Soil)

Hydroconsolidation = 0.3 %

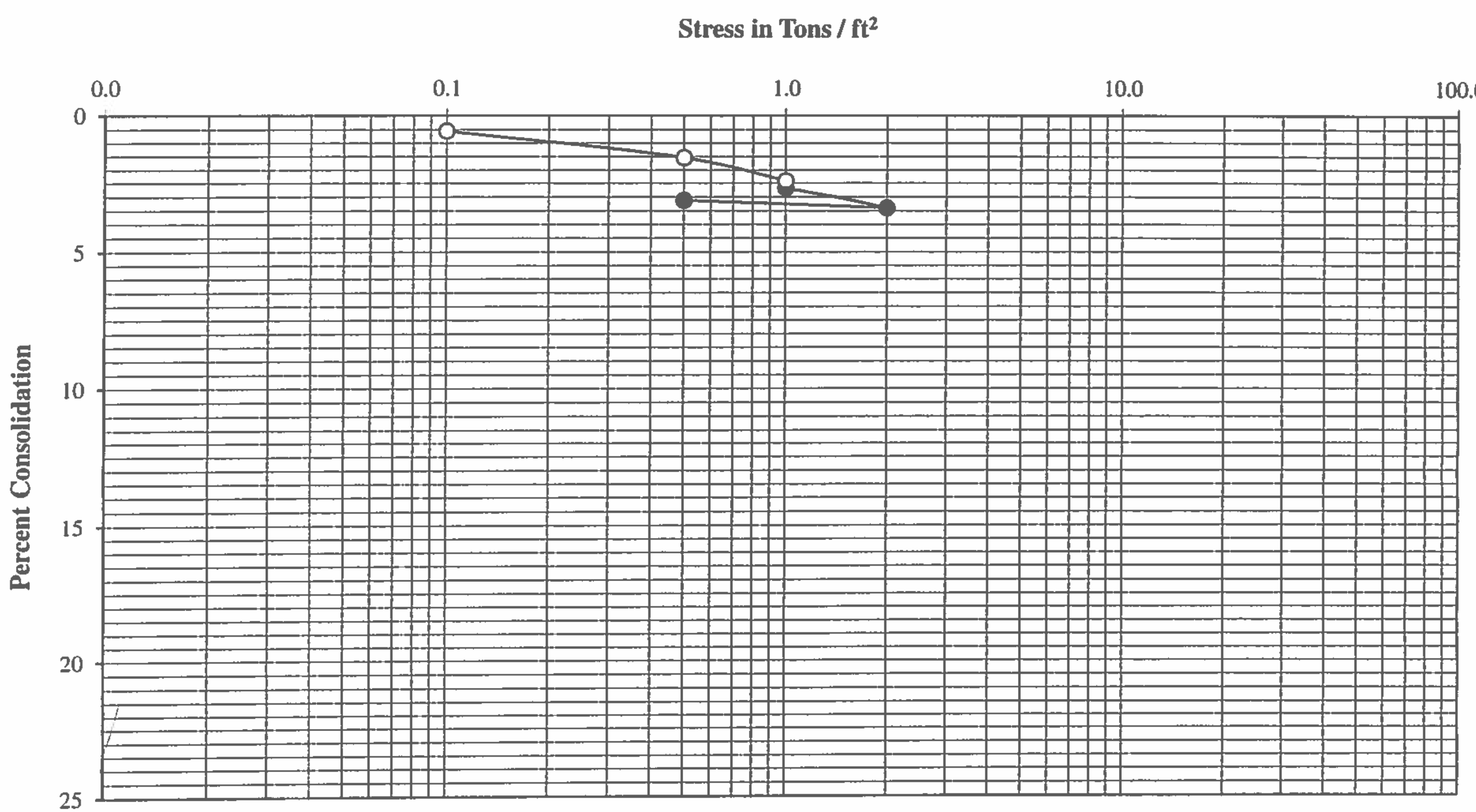


Plate CS - 14

C. Y. GEOTECH, INC.

Geotechnical Engineering and Engineering Geology

Giant Inland Empire RV Center Inc.

Date : 01-2020 | P.N. No: CYG-19-8863

Consolidation Test

Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
Onsite Soil	-- --	7.5	15	1.0	2.4

Classification : Light brown silty sand (Remolded Soil)

Hydroconsolidation = 0.3 %

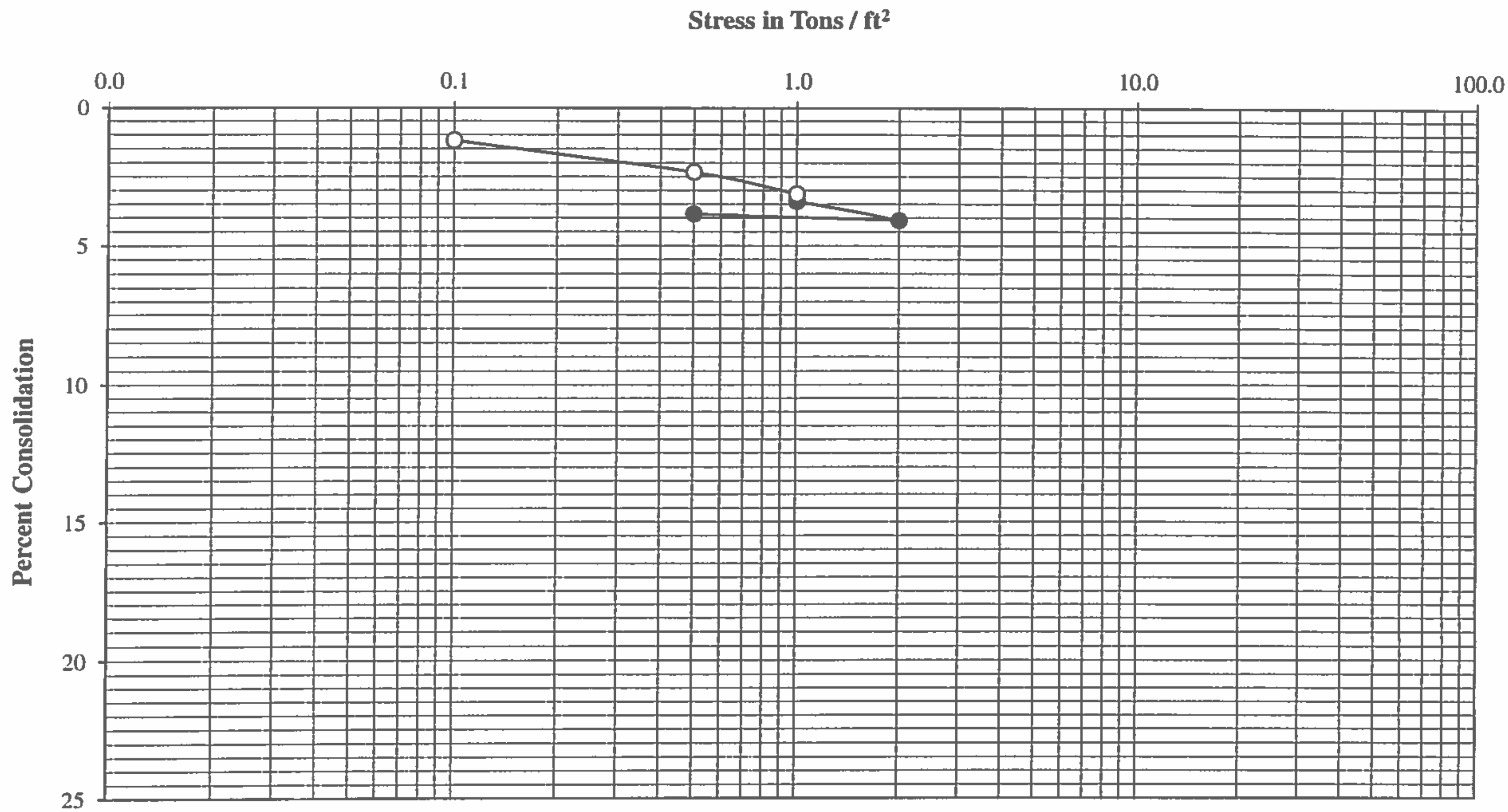
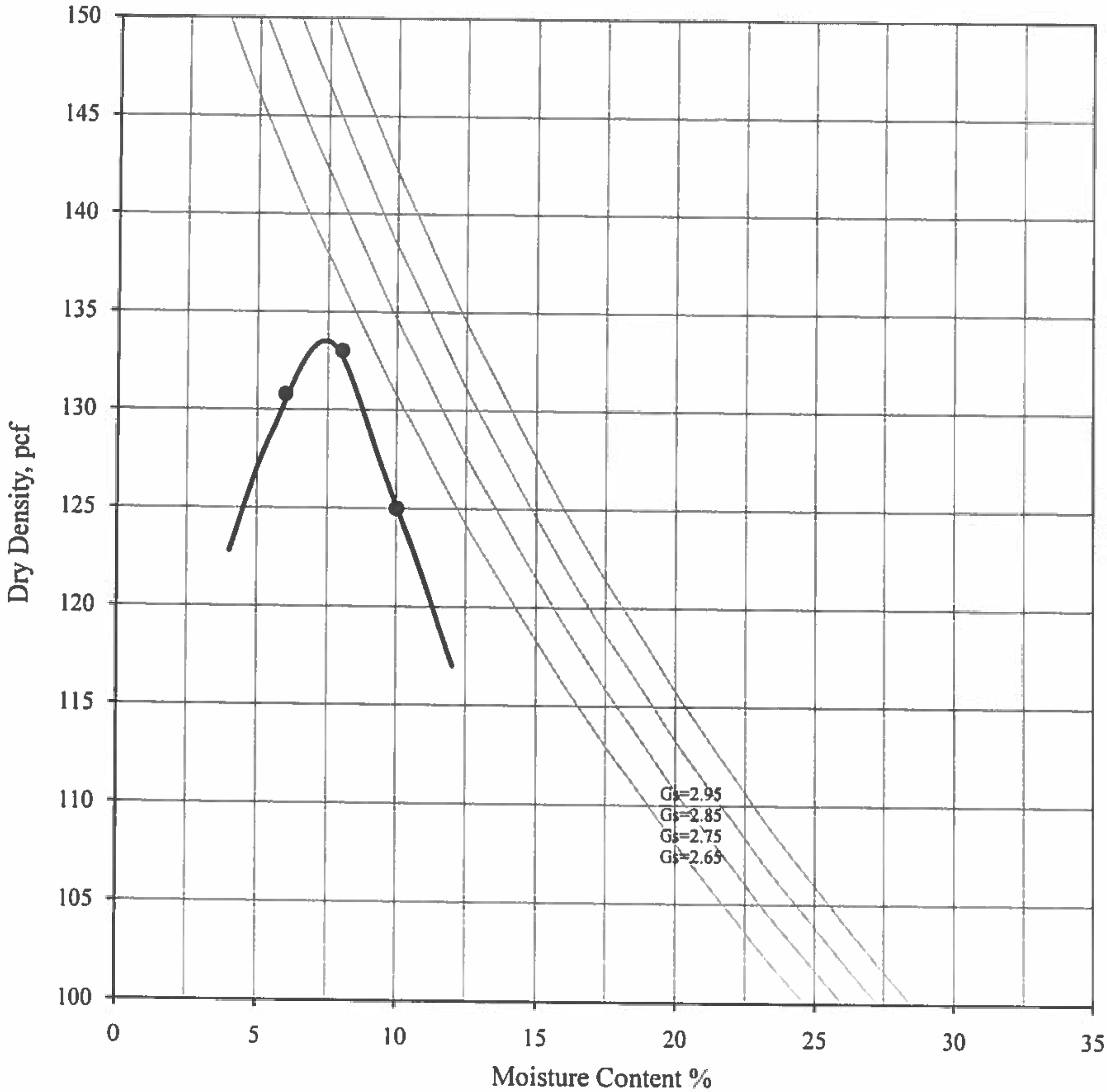


Plate CS - 15



Maximum Dry Density = 133.5 pcf

Optimum Moisture Content = 7.5 %

Boring : Onsite Mix
Depth : -- --
Description : Gray gravelly sand

C. Y. GEOTECH, INC.

Geotechnical Engineering
and Engineering Geology

Giant Inland Empire RV Center Inc.

Date : 01-2020

P.N. No.: CYG-19-8863

Compaction Curve

Plate CM - 1

C. Y. GEOTECH, INC.

Geotechnical Engineering and Engineering Geology

Giant Inland Empire RV Center Inc.

Date : 01-2020	P.N. No.: CYG-20-8863

Sieve Analysis

Boring	Depth (feet)	γ_{Field} (pcf)	$\gamma_{Saturate}$ (pcf)	Fines (%)	D_{50} (mm)
B-1	2.5	-- --	-- --	0	0.287

Classification : Grayish brown sand (Qa)

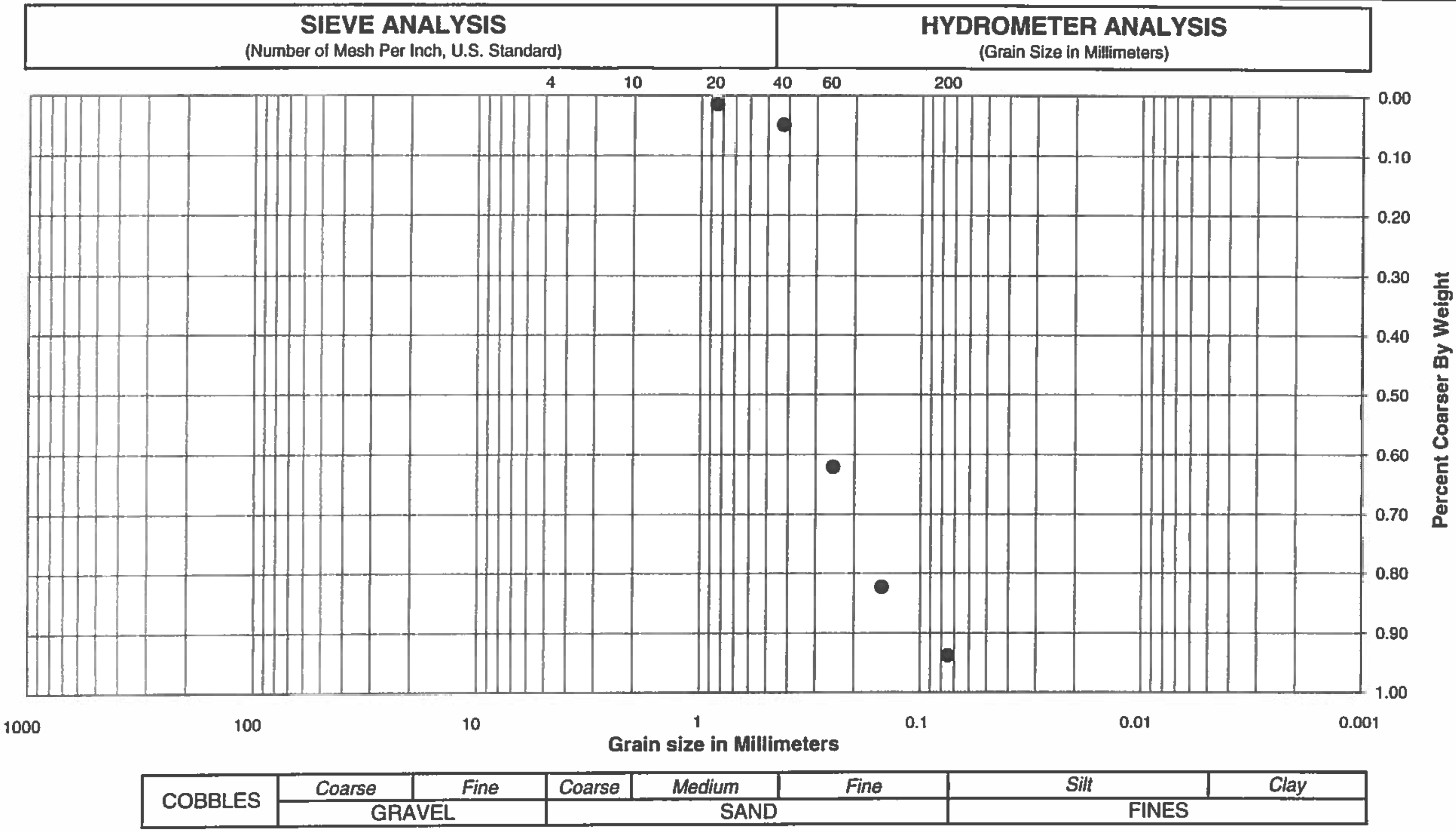


Plate GS - 1

C. Y. GEOTECH, INC.

Geotechnical Engineering and Engineering Geology

Giant Inland Empire RV Center Inc.

Date : 01-2020 | P.N. No.: CYG-20-8863

Sieve / Hydrometer Analysis

Boring	Depth (feet)	γ_{Field} (pcf)	$\gamma_{Saturate}$ (pcf)	Fines (%)	D_{50} (mm)
B-1	12.5	-- --	-- --	50	0.075

Classification : Grayish brown silty sand (Qa)

Clay Content = 5 %

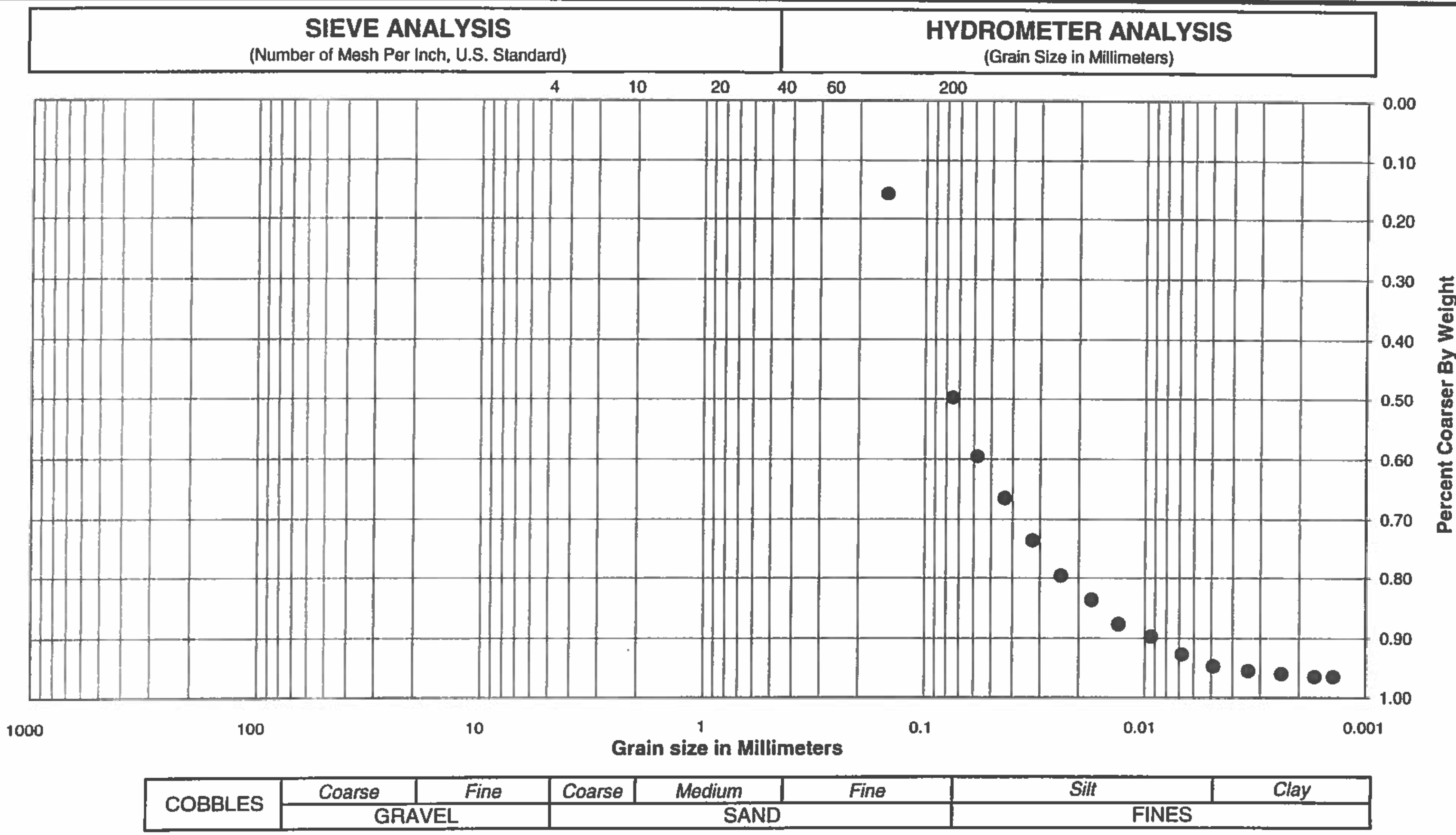


Plate GS - 2

C. Y. GEOTECH, INC.

Geotechnical Engineering and Engineering Geology

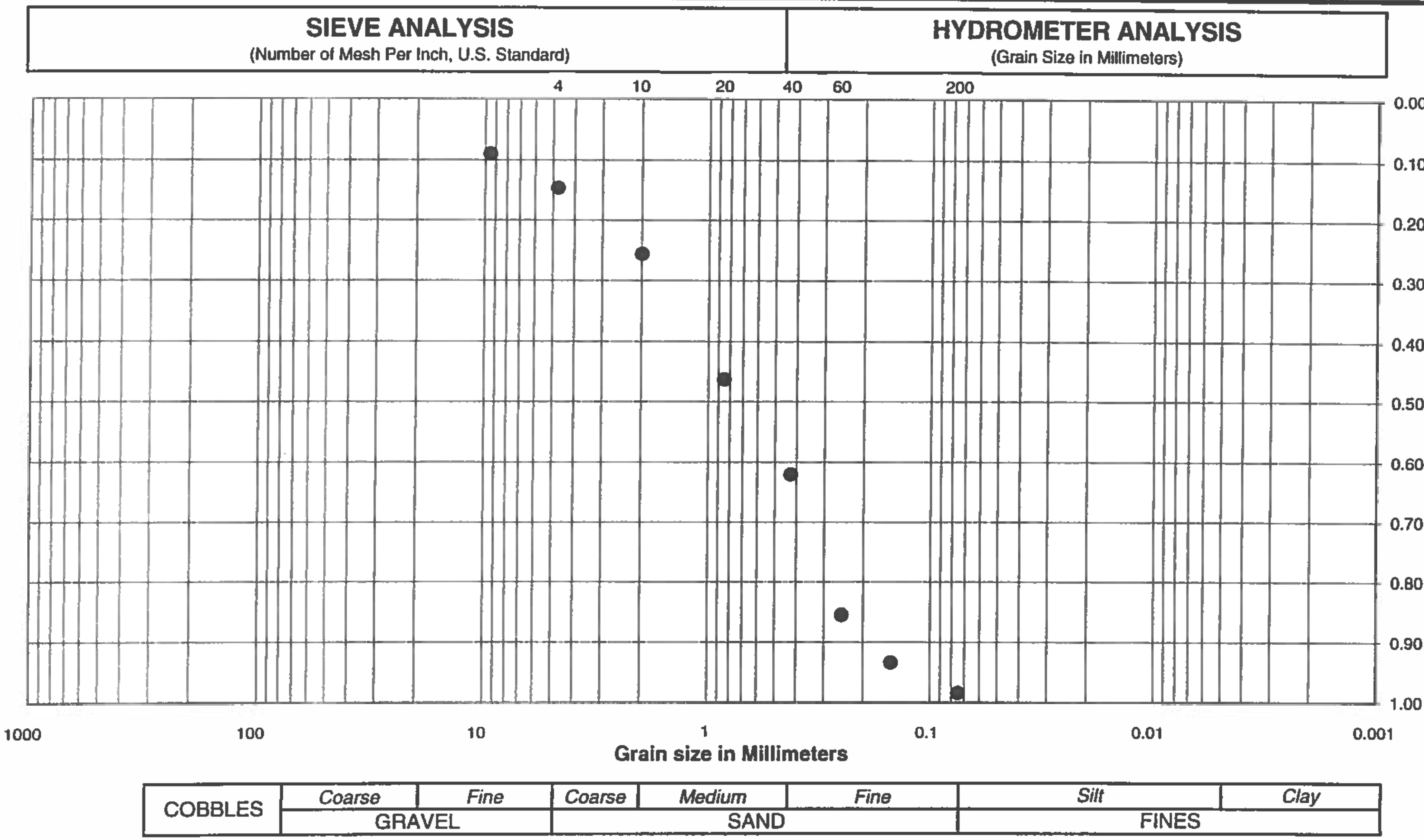
Giant Inland Empire RV Center Inc.

Date : 01-2020 | P.N. No.: CYG-20-8863

Sieve Analysis

Boring	Depth (feet)	γ_{Field} (pcf)	$\gamma_{Saturate}$ (pcf)	Fines (%)	D_{50} (mm)
B-1	27.5	-- --	-- --	0	0.749

Classification : Gray gravelly sand (Qa)



C. Y. GEOTECH, INC.

Geotechnical Engineering and Engineering Geology

Giant Inland Empire RV Center Inc.

Date : 01-2020	P.N. No.: CYG-20-8863

Sieve Analysis

Boring	Depth (feet)	γ_{Field} (pcf)	$\gamma_{Saturate}$ (pcf)	Fines (%)	D_{50} (mm)
B-1	32.5	-- --	-- --	0	3.311

Classification : Gray gravelly sand (Qa)

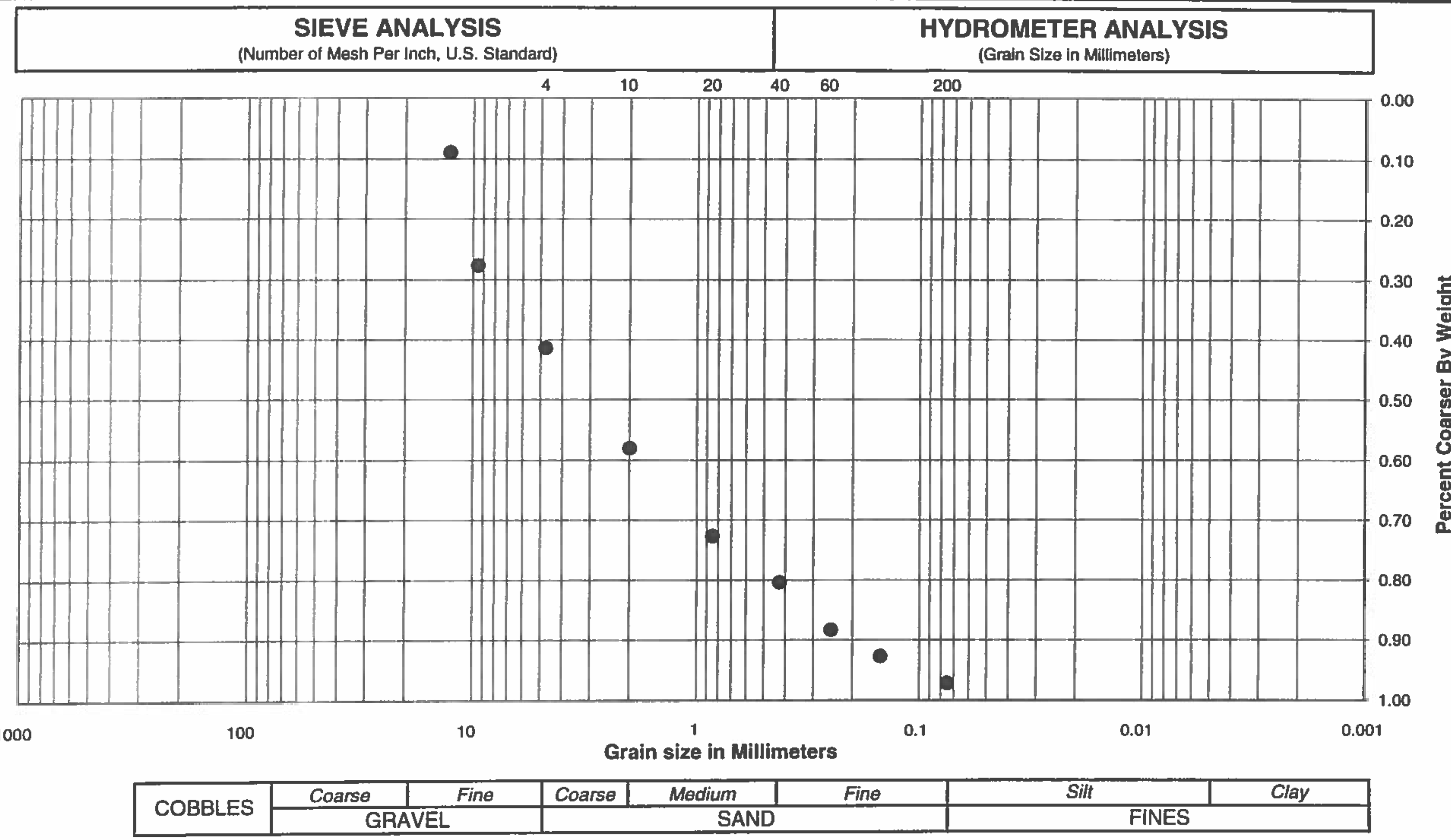


Plate GS - 4

C. Y. GEOTECH, INC.

Geotechnical Engineering and Engineering Geology

Giant Inland Empire RV Center Inc.

Date : 01-2020	P.N. No.: CYG-20-8863

Sieve Analysis

Boring	Depth (feet)	γ_{Field} (pcf)	$\gamma_{Saturate}$ (pcf)	Fines (%)	D_{50} (mm)
B-2	22.5	-- --	-- --	0	0.950

Classification : Grayish brown gravelly sand (Qa)

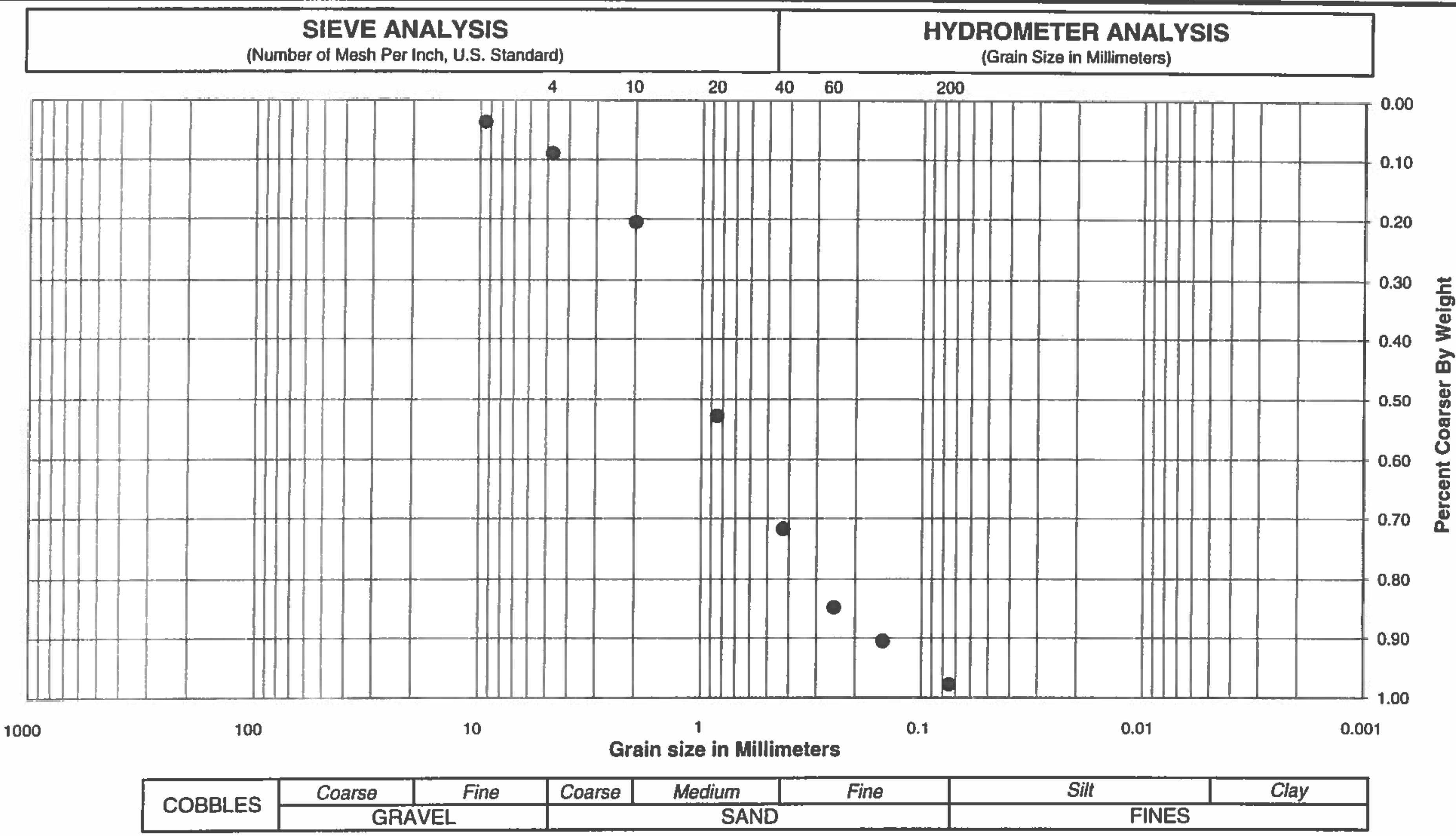


Plate GS - 5

Results Only Soil Testing
for
John Braly

January 16, 2020

Prepared for:
Paul Cai
C. Y. Geotech, Inc.
9428 Eton Ave, Unit M
Chatsworth, CA
cygeotech@sbcglobal.net

Project X Job#: S200113H
Client Job or PO#: CYG-19-8863

Respectfully Submitted,

Ed

Eduardo Hernandez, M.Sc., P.E.
Sr. Corrosion Consultant
NACE Corrosion Technologist #16592
Professional Engineer
California No. M37102
ehernandez@projectxcorrosion.com

REGISTERED PROFESSIONAL ENGINEER
EDUARDO HERNANDEZ
No. M37102
MECHANICAL
STATE OF CALIFORNIA

Soil Analysis Lab Results

Client: C. Y. Geotech, Inc.
Job Name: John Braly
Client Job Number: CYG-19-8863
Project X Job Number: S200113H
January 16, 2020

	Method	CTM 417		CTM 422		CTM 643		CTM 643
Bore# / Description	Depth	Sulfates SO_4^{2-}		Chlorides Cl^-		Resistivity		pH
						As Rec'd	Minimum	
	(ft)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(Ohm-cm)	(Ohm-cm)	
Onsite-Grayish Brown Silty	0-5	24.2	0.0024	10.8	0.0011	201,000	NT	8.2

Cations and Anions, except Sulfide and Bicarbonate, tested with Ion Chromatography
mg/kg = milligrams per kilogram (parts per million) of dry soil weight
ND = 0 = Not Detected | NT = Not Tested | Unk = Unknown
Chemical Analysis performed on 1:3 Soil-To-Water extract

29990 Technology Dr., Suite 13, Murrieta, CA 92563 Tel: 213-928-7213 Fax: 951-226-1720
www.projectxcorrosion.com

PML

Pacific Materials Laboratory, Inc.

Serving Ventura County since 1963

January 17, 2020
Lab No. 4119-5
File No. 20-5795-5

C.Y. Geotech, Inc.
21430 Strathern St., Unit #0
Canoga Park, CA 91304

SUBJECT: **R-Value Testing**
Samples Delivered to Laboratory

Gentlemen:

Pursuant to your request, R-Value testing was performed on soil samples delivered to our laboratory. R-Value testing was performed in accordance with California Test 301-F criteria. The test results follow:

R-VALUE RESULTS

PROJECT: CYG-19-8863; Giant Empire RV Center, Inc.
LOCATION: Assessor's Parcel Numbers (APN): 027614430 and 027614431

Soil Description: Light Grey Silty medium to fine Sand

ITEM	1	2	3
Compaction Pressure - psi	225/350	250/350	250/350
Initial Moisture - %	8.1	8.1	8.1
Moisture at Compaction - %	9.5	9.0	9.2
Density - pcf	124.5	125.0	125.4
R-Value	73	81	77
Exudation Pressure	263	508	316
Expansion Pressure thickness ft.	0.10	0.20	0.13

Assigned R-Value: 76*

Footnotes:
**Please verify R-value based upon expansion thickness (see California Test 301-F procedures).*

Thank you for allowing *Pacific Materials Laboratory, Inc.* to be of service. If we may be of further service regarding this or other geotechnical issues, please do not hesitate to call (805) 482-9801, fax (805) 445-6551 or write.

Respectfully Submitted,
PACIFIC MATERIALS LABORATORY, INC.

DCP:gp
Cc: Addressee (Email)

Douglas C. Papay, RCE 29,565
President

REGISTERED PROFESSIONAL ENGINEER
DOUGLAS C. PAPAY
CE 29565
Exp: 3/31/21
CIVIL
STATE OF CALIFORNIA

APPENDIX B
LIQUEFACTION EVALUATION

The liquefaction evaluation method introduced by Seed and Idriss (1985) was used in the calculation of the factors 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".

An earthquake magnitude of 7.07 based on the ASCE7-16 and a peak ground acceleration of 0.696g which equal to 2/3 PGAm (mean peak high ground acceleration) were used in liquefaction evaluations. Although no groundwater was encountered at the depth of both deep borings, the historically highest groundwater underlying the site and in site vicinity is estimated to be approximately 30 feet below the ground surface as discussed in Section 9.0. Therefore, a groundwater at a depth of 50 feet below the existing ground surface was used in the calculation of relative density, C_N value and corrected N value ($N_{1(60)}$), and a groundwater depth of 30 feet was used to calculate the earthquake induced stress ratio.

The historically highest groundwater levels, earthquake magnitudes and peak ground accelerations used in the liquefaction evaluations are listed in the following table.

Depth of Groundwater Observed in Boring, ft	Depth to Historically Highest Groundwater, ft	Maximum Credible Magnitude of Earthquake	Peak Horizontal Ground Acceleration PGAm
> 50	30	7.07	0.696g

The input data and calculations of factors of safety for liquefaction potential are shown on Tables B.1 to B.8. The liquefaction evaluation indicated that onsite soil is not susceptible to liquefaction during an earthquake with the assumed earthquake magnitude, ground accelerations and groundwater level. The results of the liquefaction evaluations are summarized in the following table.

Depth (ft)	Susceptibility to Liquefaction	Remarks
0 - 30	Not Susceptible	Above Groundwater
30 - 50	Not Susceptible	Relatively High SPT N Values

Liquefaction Analysis (Seed and Idriss Method, 1985)

Program Made by C. Y. Geotech, Inc.

Project Name: CYG-19-8863 APN: 027614430 and 027614431 Boring B-1

Input data of Liquefaction Evaluation

Depth of Groundwater Observed in Boring	=	50 ft
Depth to Historically Highest Groundwater	=	30 ft
Max. Credible Magnitude of Earthquake (10% @ 50 yrs.)	=	7.07
Peak Horizontal Ground Acceleration (2/3 PGA_M)	=	0.696 g

Scaling Factor for M = 7.07 = 1.07

Table B.1 Soil Parameters Obtained from Field and Lab Tests

Field Soil Sample Layer	Bottom Depth, ft	Field SPT N Value	Depth of SPT Sample, ft	Dry Density pcf	Moisture Content %	Field Density pcf	Saturated Density pcf	% Fine Material	% Clay Material	Saturated Moisture Content (MC_{sat}), %	Liquid Limit (LL), %	Plastic Limit (PL), %	Plasticity Index (PI)	Ratio of (MCsat) to LL, %
1	5	16	2.5	106	3	109	129	0	0	22	--	--	--	--
2	10	17	7.5	109	11	121	131	0	0	20	--	--	--	--
3	15	20	12.5	111	16	129	132	50	5	19	--	--	--	--
4	20	35	17.5	109	4	113	131	0	0	20	--	--	--	--
5	25	35	22.5	116	2	118	135	0	0	17	--	--	--	--
6	30	43	27.5	122	2	124	139	0	0	14	--	--	--	--
7	35	43	32.5	118	6	125	137	0	0	16	--	--	--	--
8	40	100	37.5	121	5	127	139	0	0	15	--	--	--	--
9	45	47	42.5	118	2	120	137	0	0	16	--	--	--	--
10	50	100	47.5	129	2	132	144	0	0	11	--	--	--	--

Table B.2 Soil Parameters Used in Liquefaction Analyses

Depth ft	Field N Value	% Fine	% Clay	Dry Density pcf	Moisture Content, %	Field Density pcf	Saturated Density pcf	Density for Effective Stress (1)	Density for Effective Stress (2)	Relative Density
2.5	16	0	0	106	3	109	129	109	109	75
5	16	0	0	106	3	109	129	109	109	75
7.5	17	0	0	109	11	131	131	131	131	68
10	17	0	0	109	11	131	131	131	131	68
12.5	20	50	5	111	16	129	132	129	129	68
15	20	50	5	111	16	129	132	129	129	68
17.5	35	0	0	109	4	113	131	113	113	84
20	35	0	0	109	4	113	131	113	113	84
22.5	35	0	0	116	2	118	135	118	118	80
25	35	0	0	116	2	118	135	118	118	80
27.5	43	0	0	122	2	139	139	124	124	84
30	43	0	0	122	2	139	139	124	124	84
32.5	43	0	0	118	6	125	137	125	137	80
35	43	0	0	118	6	125	137	125	137	80
37.5	100	0	0	121	5	127	139	127	139	116
40	100	0	0	121	5	127	139	127	139	116
42.5	47	0	0	118	2	120	137	120	137	76
45	47	0	0	118	2	120	137	120	137	76
47.5	100	0	0	129	2	132	144	132	144	107
50	100	0	0	129	2	132	144	132	144	107

(1) Effective stresses for groundwater at a depth of 50 feet (observed groundwater)

(2) Effective stresses for groundwater at a depth of 30 feet (Historical Highest Groundwater)

Table B.3 Result of Liquefaction Analyses

Soil Layer	Depth of Layer Bottom, ft	Total Stress tsf	Effective Stress (1) tsf	Effective Stress (2) tsf	Field N Value	Depth Correction	Sampler Correction Cs	Estimated Relative Density, %	CN	% Fine	N1(60) (3)	α (4)	β (4)	N1(60) CR (5)
1	2.5	0.136	0.136	0.136	16	0.75	1.2	75	2.000	0	33.1	0.00	1.00	33.1
2	5	0.273	0.273	0.273	16	0.75	1.2	75	1.874	0	31.0	0.00	1.00	31.0
3	7.5	0.436	0.436	0.436	17	0.75	1.2	68	1.453	0	25.6	0.00	1.00	25.6
4	10	0.600	0.600	0.600	17	0.75	1.2	68	1.275	0	22.4	0.00	1.00	22.4
5	12.5	0.761	0.761	0.761	20	1	1.2	68	1.142	50	31.5	5.00	1.20	42.8
6	15	0.923	0.923	0.923	20	1	1.2	68	1.045	50	28.8	5.00	1.20	39.6
7	17.5	1.064	1.064	1.064	35	1	1.2	84	0.978	0	47.2	0.00	1.00	47.2
8	20	1.205	1.205	1.205	35	1	1.2	84	0.929	0	44.9	0.00	1.00	44.9
9	22.5	1.353	1.353	1.353	35	1	1.2	80	0.884	0	42.7	0.00	1.00	42.7
10	25	1.500	1.500	1.500	35	1	1.2	80	0.848	0	41.0	0.00	1.00	41.0
11	27.5	1.655	1.655	1.655	43	1	1.2	84	0.814	0	48.3	0.00	1.00	48.3
12	30	1.810	1.810	1.810	43	1	1.2	84	0.783	0	46.5	0.00	1.00	46.5
13	32.5	1.981	1.981	1.903	43	1	1.2	80	0.749	0	44.5	0.00	1.00	44.5
14	35	2.153	2.153	1.997	43	1	1.2	80	0.718	0	42.6	0.00	1.00	42.6
15	37.5	2.326	2.326	2.092	100	1	1.2	116	0.691	0	95.4	0.00	1.00	95.4
16	40	2.500	2.500	2.188	100	1	1.2	116	0.668	0	92.2	0.00	1.00	92.2
17	42.5	2.671	2.671	2.281	47	1	1.2	76	0.646	0	41.9	0.00	1.00	41.9
18	45	2.843	2.843	2.375	47	1	1.2	76	0.626	0	40.6	0.00	1.00	40.6
19	47.5	3.023	3.023	2.477	100	1	1.2	107	0.608	0	83.9	0.00	1.00	83.9
20	50	3.203	3.203	2.579	100	1	1.2	107	0.589	0	81.3	0.00	1.00	81.3

(1) Effective stresses for groundwater at a depth of 50 feet (observed groundwater)

(2) Effective stresses for groundwater at a depth of 30 feet (Historical Highest Groundwater)

(3) N1(60) = N x CN x Cs x Cb x 0.75 when depth ≤ 10 ft   N 1(60) = N x CN x Cs x Cb when depth > 10 ft   A Cs value of 1.2 was used for sampler with no liner   A Cb value of 1.15 was used for borehole correction

(4) Coefficients α and β were considered for correction of (N1)60 by the influence of fines content

Table B.4 Result of Liquefaction Analyses

Soil Layer	Depth of Layer Bottom, ft	Total Stress tsf	Effective Stress (1) tsf	Effective Stress (2) tsf	Ks	Rd	Earthquake Magnitude	Peak Ground Acceleration (g)	Scaling Factor	Liquefaction Stress Ratio (3)	EQ Induced Stress Ratio	Factor of Safety	Liquefiable (Yes/No)
1	2.5	0.136	0.136	0.136	1	0.995	7.07	0.696	1.07	AG	0.421	----	No
2	5	0.273	0.273	0.273	1	0.993	7.07	0.696	1.07	AG	0.420	—	No
3	7.5	0.436	0.436	0.436	1	0.990	7.07	0.696	1.07	AG	0.419	—	No
4	10	0.600	0.600	0.600	1	0.980	7.07	0.696	1.07	AG	0.414	—	No
5	12.5	0.761	0.761	0.761	1	0.975	7.07	0.696	1.07	AG	0.412	----	No
6	15	0.923	0.923	0.923	1	0.970	7.07	0.696	1.07	AG	0.410	----	No
7	17.5	1.064	1.064	1.064	0.997	0.965	7.07	0.696	1.07	AG	0.408	—	No
8	20	1.205	1.205	1.205	0.992	0.960	7.07	0.696	1.07	AG	0.406	----	No
9	22.5	1.353	1.353	1.353	0.986	0.950	7.07	0.696	1.07	AG	0.402	----	No
10	25	1.500	1.500	1.500	0.98	0.940	7.07	0.696	1.07	AG	0.397	—	No
11	27.5	1.655	1.655	1.655	0.971	0.930	7.07	0.696	1.07	AG	0.393	—	No
12	30	1.810	1.810	1.810	0.963	0.920	7.07	0.696	1.07	AG	0.389	—	No
13	32.5	1.981	1.981	1.903	0.953	0.910	7.07	0.696	1.07	Infinite	0.401	Infinite	No
14	35	2.153	2.153	1.997	0.942	0.890	7.07	0.696	1.07	Infinite	0.406	Infinite	No
15	37.5	2.326	2.326	2.092	0.93	0.860	7.07	0.696	1.07	Infinite	0.404	Infinite	No
16	40	2.500	2.500	2.188	0.918	0.840	7.07	0.696	1.07	Infinite	0.406	Infinite	No
17	42.5	2.671	2.671	2.281	0.907	0.820	7.07	0.696	1.07	Infinite	0.406	Infinite	No
18	45	2.843	2.843	2.375	0.896	0.800	7.07	0.696	1.07	Infinite	0.405	Infinite	No
19	47.5	3.023	3.023	2.477	0.884	0.780	7.07	0.696	1.07	Infinite	0.402	Infinite	No
20	50	3.203	3.203	2.579	0.871	0.760	7.07	0.696	1.07	Infinite	0.399	Infinite	No

(1) Effective stresses for groundwater at a depth of 50 feet (observed groundwater)

(2) Effective stresses for groundwater at a depth of 30 feet (Historical Highest Groundwater)

(3) $N1(60) = N \times CN \times Cs \times Cb \times 0.75$ when depth ≤ 10 ft $N1(60) = N \times CN \times Cs \times Cb$ when depth > 10 ft   A Cs value of 1.2 was used for sampler with no liner   A Cb value of 1.15 was used for borehole correction

(4) AG : soil is non-liquefiable due to above groundwater

(5) HPI : soil is non-liquefiable due to high plasticity index (PI>18)

(6) RML : soil is non-liquefiable due to low ratio of Saturated Moisture Content (Mcsat,%) to Liquid Limit (LL,%) (RML < 80)

Magnitude of earthquake = 7.07   Scaling factor = 1.07

Peak horizontal ground acceleration = 0.696g

Liquefaction Analysis (Seed and Idriss Method, 1985)

Program Made by C. Y. Geotech, Inc.

Project Name: CYG-19-8863 APN: 027614430 and 027614431 Boring B-2

Input data of Liquefaction Evaluation

Depth of Groundwater Observed in Boring	=	50 ft
Depth to Historically Highest Groundwater	=	30 ft
Max. Credible Magnitude of Earthquake (10% @ 50 yrs.)	=	7.07
Peak Horizontal Ground Acceleration (2/3 PGA_M)	=	0.696 g
Scaling Factor for M = 7.07	=	1.07

Table B.5 Soil Parameters Obtained from Field and Lab Tests

Field Soil Sample Layer	Bottom Depth, ft	Field SPT N Value	Depth of SPT Sample, ft	Dry Density pcf	Moisture Content %	Field Density pcf	Saturated Density pcf	% Fine Material	% Clay Material	Saturated Moisture Content (MC_{sat}), %	Liquid Limit (LL), %	Plastic Limit (PL), %	Plasticity Index (PI)	Ratio of (MCsat) to LL, %
1	5	12	2.5	110	5	116	132	0	0	20	--	--	--	--
2	10	16	7.5	110	18	130	132	0	0	20	--	--	--	--
3	15	22	12.5	111	1	112	132	0	0	19	--	--	--	--
4	20	35	17.5	136	4	141	148	0	0	9	--	--	--	--
5	25	32	22.5	117	6	124	136	0	0	16	--	--	--	--
6	30	37	27.5	126	5	132	142	0	0	12	--	--	--	--
7	35	100	32.5	117	5	123	136	0	0	16	--	--	--	--
8	40	52	37.5	125	7	134	141	0	0	13	--	--	--	--
9	45	100	42.5	125	5	131	141	0	0	13	--	--	--	--
10	50	86	47.5	127	3	131	142	0	0	12	--	--	--	--

Table B.6 Soil Parameters Used in Liquefaction Analyses

Depth ft	Field N Value	% Fine	% Clay	Dry Density pcf	Moisture Content, %	Field Density pcf	Saturated Density pcf	Density for Effective Stress (1)	Density for Effective Stress (2)	Relative Density
2.5	12	0	0	110	5	116	132	116	116	64
5	12	0	0	110	5	116	132	116	116	64
7.5	16	0	0	110	18	130	132	130	130	66
10	16	0	0	110	18	130	132	130	130	66
12.5	22	0	0	111	1	112	132	112	112	71
15	22	0	0	111	1	112	132	112	112	71
17.5	35	0	0	136	4	141	148	141	141	83
20	35	0	0	136	4	141	148	141	141	83
22.5	32	0	0	117	6	124	136	124	124	75
25	32	0	0	117	6	124	136	124	124	75
27.5	37	0	0	126	5	142	142	132	132	77
30	37	0	0	126	5	142	142	132	132	77
32.5	100	0	0	117	5	123	136	123	136	120
35	100	0	0	117	5	123	136	123	136	120
37.5	52	0	0	125	7	134	141	134	141	83
40	52	0	0	125	7	134	141	134	141	83
42.5	100	0	0	125	5	131	141	131	141	111
45	100	0	0	125	5	131	141	131	141	111
47.5	86	0	0	127	3	131	142	131	142	98
50	86	0	0	127	3	131	142	131	142	98

(1) Effective stresses for groundwater at a depth of 50 feet (observed groundwater)

(2) Effective stresses for groundwater at a depth of 30 feet (Historical Highest Groundwater)

Table B.7 Result of Liquefaction Analyses

Soil Layer	Depth of Layer Bottom, ft	Total Stress tsf	Effective Stress (1) tsf	Effective Stress (2) tsf	Field N Value	Depth Correction	Sampler Correction Cs	Estimated Relative Density, %	CN	% Fine	N1(60) (3)	α (4)	β (4)	N1(60) CR (5)
1	2.5	0.145	0.145	0.145	12	0.75	1.2	64	2.000	0	24.8	0.00	1.00	24.8
2	5	0.290	0.290	0.290	12	0.75	1.2	64	1.776	0	22.1	0.00	1.00	22.1
3	7.5	0.453	0.453	0.453	16	0.75	1.2	66	1.432	0	23.7	0.00	1.00	23.7
4	10	0.615	0.615	0.615	16	0.75	1.2	66	1.262	0	20.9	0.00	1.00	20.9
5	12.5	0.755	0.755	0.755	22	1	1.2	71	1.147	0	34.8	0.00	1.00	34.8
6	15	0.895	0.895	0.895	22	1	1.2	71	1.060	0	32.2	0.00	1.00	32.2
7	17.5	1.071	1.071	1.071	35	1	1.2	83	0.975	0	47.1	0.00	1.00	47.1
8	20	1.248	1.248	1.248	35	1	1.2	83	0.916	0	44.2	0.00	1.00	44.2
9	22.5	1.403	1.403	1.403	32	1	1.2	75	0.869	0	38.4	0.00	1.00	38.4
10	25	1.558	1.558	1.558	32	1	1.2	75	0.835	0	36.9	0.00	1.00	36.9
11	27.5	1.723	1.723	1.723	37	1	1.2	77	0.801	0	40.9	0.00	1.00	40.9
12	30	1.888	1.888	1.888	37	1	1.2	77	0.768	0	39.2	0.00	1.00	39.2
13	32.5	2.058	2.058	1.980	100	1	1.2	120	0.735	0	101.4	0.00	1.00	101.4
14	35	2.228	2.228	2.072	100	1	1.2	120	0.706	0	97.4	0.00	1.00	97.4
15	37.5	2.404	2.404	2.170	52	1	1.2	83	0.680	0	48.8	0.00	1.00	48.8
16	40	2.580	2.580	2.268	52	1	1.2	83	0.658	0	47.2	0.00	1.00	47.2
17	42.5	2.756	2.756	2.366	100	1	1.2	111	0.635	0	87.6	0.00	1.00	87.6
18	45	2.933	2.933	2.465	100	1	1.2	111	0.617	0	85.2	0.00	1.00	85.2
19	47.5	3.110	3.110	2.564	86	1	1.2	98	0.598	0	71.0	0.00	1.00	71.0
20	50	3.288	3.288	2.664	86	1	1.2	98	0.582	0	69.1	0.00	1.00	69.1

(1) Effective stresses for groundwater at a depth of 50 feet (observed groundwater)

(2) Effective stresses for groundwater at a depth of 30 feet (Historical Highest Groundwater)

(3) N1(60) = N x CN x Cs x Cb x 0.75 when depth ≤ 10 ft N 1(60) = N x CN x Cs x Cb when depth > 10 ft A Cs value of 1.2 was used for sampler with no liner A Cb value of 1.15 was used for borehole correction

(4) Coefficients α and β were considered for correction of (N1)60 by the influence of fines content

Table B.8 Result of Liquefaction Analyses

Soil Layer	Depth of Layer Bottom, ft	Total Stress tsf	Effective Stress (1) tsf	Effective Stress (2) tsf	Ks	Rd	Earthquake Magnitude	Peak Ground Acceleration (g)	Scaling Factor	Liquefaction Stress Ratio (3)	EQ Induced Stress Ratio	Factor of Safety	Liquefiable (Yes/No)
1	2.5	0.145	0.145	0.145	1	0.995	7.07	0.696	1.07	AG	0.421	—	No
2	5	0.290	0.290	0.290	1	0.993	7.07	0.696	1.07	AG	0.420	—	No
3	7.5	0.453	0.453	0.453	1	0.990	7.07	0.696	1.07	AG	0.419	—	No
4	10	0.615	0.615	0.615	1	0.980	7.07	0.696	1.07	AG	0.414	—	No
5	12.5	0.755	0.755	0.755	1	0.975	7.07	0.696	1.07	AG	0.412	—	No
6	15	0.895	0.895	0.895	1	0.970	7.07	0.696	1.07	AG	0.410	—	No
7	17.5	1.071	1.071	1.071	0.997	0.965	7.07	0.696	1.07	AG	0.408	—	No
8	20	1.248	1.248	1.248	0.99	0.960	7.07	0.696	1.07	AG	0.406	—	No
9	22.5	1.403	1.403	1.403	0.984	0.950	7.07	0.696	1.07	AG	0.402	—	No
10	25	1.558	1.558	1.558	0.977	0.940	7.07	0.696	1.07	AG	0.397	—	No
11	27.5	1.723	1.723	1.723	0.968	0.930	7.07	0.696	1.07	AG	0.393	—	No
12	30	1.888	1.888	1.888	0.958	0.920	7.07	0.696	1.07	AG	0.389	—	No
13	32.5	2.058	2.058	1.980	0.948	0.910	7.07	0.696	1.07	Infinite	0.400	Infinite	No
14	35	2.228	2.228	2.072	0.937	0.890	7.07	0.696	1.07	Infinite	0.405	Infinite	No
15	37.5	2.404	2.404	2.170	0.925	0.860	7.07	0.696	1.07	Infinite	0.403	Infinite	No
16	40	2.580	2.580	2.268	0.913	0.840	7.07	0.696	1.07	Infinite	0.404	Infinite	No
17	42.5	2.756	2.756	2.366	0.902	0.820	7.07	0.696	1.07	Infinite	0.404	Infinite	No
18	45	2.933	2.933	2.465	0.89	0.800	7.07	0.696	1.07	Infinite	0.402	Infinite	No
19	47.5	3.110	3.110	2.564	0.878	0.780	7.07	0.696	1.07	Infinite	0.400	Infinite	No
20	50	3.288	3.288	2.664	0.865	0.760	7.07	0.696	1.07	Infinite	0.397	Infinite	No

(1) Effective stresses for groundwater at a depth of 50 feet (observed groundwater)

(2) Effective stresses for groundwater at a depth of 30 feet (Historical Highest Groundwater)

(3) N1(60) = N x CN x Cs x Cb x 0.75 when depth ≤ 10 ft   N 1(60) = N x CN x Cs x Cb when depth > 10 ft   A Cs value of 1.2 was used for sampler with no liner   A Cb value of 1.15 was used for borehole correction

(4) AG : soil is non-liquefiable due to above groundwater

(5) HPI : soil is non-liquefiable due to high plasticity index (PI>18)

(6) RML : soil is non-liquefiable due to low ratio of Saturated Moisture Content (Mcsat,%) to Liquid Limit (LL,%) (RML < 80)

Magnitude of earthquake = 7.07   Scaling factor = 1.07

Peak horizontal ground acceleration = 0.696g

APPENDIX C

INFILTRATION TEST DATA AND CALCULATION

Percolation Test Data Sheet

Project Name	Giant Inland Empire RV Center Inc. / Colton - Santo Antonio	Project No.	CYG-19-8863
Date	01-13-20		
Test Hole No.	B-3	Tested by:	Paul Cai
Depth of Test Hole (Dr)	10 ft below ground	USCS Soil Classification	SP
Diameter of Rounded Test Hole (inch)	Diameter : 8 inches		
Dimension of Rectangular Test Hole (inch)	Length : N/A	Width : N/A	

Trial No.	Start Time	Stop Time	Time Interval (min)	Initial Depth to Water (inch)	Final Depth to Water (inch)	Change in Water Level (inch)	Greater than or Equal to 6 inches (Yes/No)
1	13:38pm	14:03pm	25	72	120	48	Yes
2	14:04pm	14:29pm	25	72	120	48	Yes

* If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soaking (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute interval) with a precision of at least 0.25 inch.

Trial No.	Start Time	Stop Time	Time Interval (min)	Initial Depth to Water (inch)	Final Depth to Water (inch)	Change in Water Level (inch)	Infiltration Rate with No Reduction Factor (in/min)	Infiltration Rate with Reduction Factor (in/hr)	Infiltration Rate with Factor of Safety = 2 (in/hr)
1	14:30pm	14:40pm	10	78	115.2	37.2	3.72	17.6	8.8
2	14:41pm	14:51pm	10	78	115.2	37.2	3.72	17.6	8.8
3	14:52pm	15:02pm	10	78	115.2	37.2	3.72	17.6	8.8
4	15:03pm	15:13pm	10	78	115.2	37.2	3.72	17.6	8.8
5	15:14pm	15:24pm	10	78	115.2	37.2	3.72	17.6	8.8
6	15:25pm	15:35pm	10	78	115.2	37.2	3.72	17.6	8.8

Comments: Recommended Infiltration Rate = 8.8 in/hr

Percolation Test Data Sheet

Project Name	Giant Inland Empire RV Center Inc. / Colton - Santo Antonio	Project No.	CYG-19-8863	Date	01-13-20
Test Hole No.	B-4	Tested by:	Paul Cai / Juan		
Depth of Test Hole (Dr)	10 ft below ground	USCS Soil Classification	SP		
Diameter of Rounded Test Hole (inch)	Diameter : 8 inches				
Dimension of Rectangular Test Hole (inch)	Length : N/A	Width : N/A			

Trial No.	Start Time	Stop Time	Time Interval (min)	Initial Depth to Water (inch)	Final Depth to Water (inch)	Change in Water Level (inch)	Greater than or Equal to 6 inches (Yes/No)
1	12:37pm	13:02pm	25	72	120	48	Yes
2	13:03pm	13:28pm	25	72	120	48	Yes

* If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soaking (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute interval) with a precision of at least 0.25 inch.

Trial No.	Start Time	Stop Time	Time Interval (min)	Initial Depth to Water (inch)	Final Depth to Water (inch)	Change in Water Level (inch)	Infiltration Rate with No Reduction Factor (in/min)	Infiltration Rate with Reduction Factor (in/hr)	Infiltration Rate with Factor of Safety = 2 (in/hr)
1	13:30pm	13:40pm	10	78	110.4	32.4	3.24	14.0	7.0
2	13:41pm	13:51pm	10	78	112.8	34.8	3.48	15.7	7.8
3	13:52pm	14:02pm	10	78	109.2	31.2	3.12	13.2	6.6
4	14:03pm	14:13pm	10	78	112.8	34.8	3.48	15.7	7.8
5	14:14pm	14:24pm	10	78	115.2	37.2	3.72	17.6	8.8
6	14:25pm	14:35pm	10	78	112.8	34.8	3.48	15.7	7.8

Comments: Recommended Infiltration Rate = 6.6 in/hr

Percolation Test Data Sheet

Project Name	Giant Inland Empire RV Center Inc. / Colton - Santo Antonio	Project No.	CYG-19-8863	Date	01-13-20
Test Hole No.	B-5	Tested by:	Paul Cai / Juan		
Depth of Test Hole (Dr)	10 ft below ground	USCS Soil Classification	SP		
Diameter of Rounded Test Hole (inch)	Diameter : 8 inches				
Dimension of Rectangular Test Hole (inch)	Length : N/A	Width : N/A			

Trial No.	Start Time	Stop Time	Time Interval (min)	Initial Depth to Water (inch)	Final Depth to Water (inch)	Change in Water Level (inch)	Greater than or Equal to 6 inches (Yes/No)
1	12:40pm	13:05pm	25	72	120	48	Yes
2	13:06pm	13:31pm	25	72	120	48	Yes

* If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soaking (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute interval) with a precision of at least 0.25 inch.

Trial No.	Start Time	Stop Time	Time Interval (min)	Initial Depth to Water (inch)	Final Depth to Water (inch)	Change in Water Level (inch)	Infiltration Rate with No Reduction Factor (in/min)	Infiltration Rate with Reduction Factor (in/hr)	Infiltration Rate with Factor of Safety = 2 (in/hr)
1	13:35pm	13:45pm	10	78	115.2	37.2	3.72	17.6	8.8
2	13:46pm	13:56pm	10	78	115.2	37.2	3.72	17.6	8.8
3	13:57pm	14:07pm	10	78	115.2	37.2	3.72	17.6	8.8
4	14:08pm	14:18pm	10	78	115.2	37.2	3.72	17.6	8.8
5	14:19pm	14:29pm	10	78	115.2	37.2	3.72	17.6	8.8
6	14:30pm	14:40pm	10	78	115.2	37.2	3.72	17.6	8.8

Comments: Recommended Infiltration Rate = 8.8 in/hr

Percolation Test Data Sheet

Project Name	Giant Inland Empire RV Center Inc. / Colton - Santo Antonio	Project No.	CYG-19-8863	Date	01-13-20
Test Hole No.	B-9	Tested by:	Paul Cai / Juan		
Depth of Test Hole (Dr)	10 ft below ground	USCS Soil Classification	SP		
Diameter of Rounded Test Hole (inch)	Diameter : 8 inches				
Dimension of Rectangular Test Hole (inch)	Length : N/A	Width : N/A			

Trial No.	Start Time	Stop Time	Time Interval (min)	Initial Depth to Water (inch)	Final Depth to Water (inch)	Change in Water Level (inch)	Greater than or Equal to 6 inches (Yes/No)
1	10:46am	11:11am	25	72	120	48	Yes
2	11:13am	11:38am	25	72	120	48	Yes

* If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soaking (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute interval) with a precision of at least 0.25 inch.

Trial No.	Start Time	Stop Time	Time Interval (min)	Initial Depth to Water (inch)	Final Depth to Water (inch)	Change in Water Level (inch)	Infiltration Rate with No Reduction Factor (in/min)	Infiltration Rate with Reduction Factor (in/hr)	Infiltration Rate with Factor of Safety = 2 (in/hr)
1	11:40am	11:50am	10	78	109.2	31.2	3.12	13.2	6.6
2	11:51am	12:01pm	10	78	115.2	37.2	3.72	17.6	8.8
3	12:02pm	12:12pm	10	78	108.0	30.0	3.00	12.4	6.2
4	12:13pm	12:23pm	10	78	105.6	27.6	2.76	10.9	5.4
5	12:24pm	12:34pm	10	78	109.2	31.2	3.12	13.2	6.6
6	12:35pm	12:45pm	10	78	111.6	33.6	3.36	14.8	7.4

Comments: Recommended Infiltration Rate = 5.4 in/hr

Percolation Test Data Sheet

Project Name	Giant Inland Empire RV Center Inc. / Colton - Santo Antonio	Project No.	CYG-19-8863	Date	01-13-20
Test Hole No.	B-10	Tested by:	Paul Cai		
Depth of Test Hole (Dr)	10 ft below ground	USCS Soil Classification	SP		
Diameter of Rounded Test Hole (inch)	Diameter : 8 inches				
Dimension of Rectangular Test Hole (inch)	Length : N/A	Width : N/A			

Trial No.	Start Time	Stop Time	Time Interval (min)	Initial Depth to Water (inch)	Final Depth to Water (inch)	Change in Water Level (inch)	Greater than or Equal to 6 inches (Yes/No)
1	10:45am	11:10am	25	72	120	48	Yes
2	11:11am	11:36am	25	72	120	48	Yes

* If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soaking (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute interval) with a precision of at least 0.25 inch.

Trial No.	Start Time	Stop Time	Time Interval (min)	Initial Depth to Water (inch)	Final Depth to Water (inch)	Change in Water Level (inch)	Infiltration Rate with No Reduction Factor (in/min)	Infiltration Rate with Reduction Factor (in/hr)	Infiltration Rate with Factor of Safety = 2 (in/hr)
1	11:37am	11:47am	10	78	115.2	37.2	3.72	17.6	8.8
2	11:48am	11:58am	10	78	109.2	31.2	3.12	13.2	6.6
3	11:59am	12:09pm	10	78	110.4	32.4	3.24	14.0	7.0
4	12:10pm	12:20pm	10	78	110.8	32.8	3.28	14.3	7.1
5	12:21pm	12:31pm	10	78	108.0	30.0	3.00	12.4	6.2
6	12:32pm	12:42pm	10	78	106.8	28.8	2.88	11.7	5.8

Comments: Recommended Infiltration Rate = 5.8 in/hr

APPENDIX D

SETTLEMENT CALCULATION

```
***********************
* SETTLEMENT ANALYSIS *
***********************
```

PROJECT NAME : CYG-19-8863 Square Ftg 2' x 2' / Q = 2000 psf / D = 2'

TYPE OF FOOTING : SQUARE FOOTING

GEOMETRY OF FOOTING :
  DEFINE DEPTH OF ORIGINAL GROUND SURFACE AS 0 FT
  RELATIVE DEPTH TO BOTTOM OF FOOTING = 2 FT.
  THICKNESS OF BACKFILL ABOVE BOTTOM OF FOOTING = 2 FT.
  THICKNESS OF COMPACTED FILL BELOW FOOTING = 0 FT.
  SIDE LENGTH OF FOOTING = 2 FT.

SOIL LAYER NO. 1
  UNIT WEIGHT = 116 PCF
  DEPTH OF LAYER BOTTOM = 30 FT.

UNIT WEIGHT OF BACKFILL = 116 PCF

APPLIED LOAD DUE TO PROPOSED STRUCTURE = 8 KIPS
APPLIED STRESS DUE TO PROPOSED STRUCTURE = 1 TSF
APPLIED STRESS DUE TO BACKFILL = .116 TSF
TOTAL APPLIED STRESS = 1.116 TSF
NET APPLIED STRESS = .9999999 TSF

SETTLEMENT OCCURS FROM 2 FT BELOW ORIGINAL GROUND SURFACE
ESTIMATED SETTLEMENT OF SEGMENT NO. 1 = .192 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 2 = 9.899999E-02 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 3 = 3.299999E-02 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 4 = .012 IN.

TOTAL ESTIMATED SETTLEMENT = .336 IN.

DISTRIBUTION OF VERTICAL EARTH PRESSURE

D : DEPTH DOWN FROM ORIGINAL GROUND SURFACE (FT.)
VP : VERTICAL EARTH PRESSURE BEFORE FOUNDATION EXCAVATION (TSF)
IND : COEFFICIENT OF APPLIED STRESS DISTRIBUTION
IVP : INDUCED VERTICAL EARTH PRESSURE (TSF)
FVP : VERTICAL EARTH PRESSURE AFTER CONSTRUCTION (TSF)

D	VP	IND	IVP	FVP	SEGMENT NO.
2	.116	1	1.116	1.116	SEGMENT 1
3	.174	.7080709	.790207	.8482071	SEGMENT 1
4	.232	.3391858	.3785313	.4945314	SEGMENT 2
5	.29	.1799187	.2007893	.3747893	SEGMENT 3
6	.348	.1084381	.1210169	.3530169	SEGMENT 4

```
***********************
* SETTLEMENT ANALYSIS *
***********************
```

PROJECT NAME : CYG-19-8863 Square Ftg 3' x 3' / Q = 2400 psf / D = 2'

TYPE OF FOOTING : SQUARE FOOTING

GEOMETRY OF FOOTING :
  DEFINE DEPTH OF ORIGINAL GROUND SURFACE AS 0 FT
  RELATIVE DEPTH TO BOTTOM OF FOOTING = 2 FT.
  THICKNESS OF BACKFILL ABOVE BOTTOM OF FOOTING = 2 FT.
  THICKNESS OF COMPACTED FILL BELOW FOOTING = 0 FT.
  SIDE LENGTH OF FOOTING = 3 FT.

SOIL LAYER NO. 1
  UNIT WEIGHT = 116 PCF
  DEPTH OF LAYER BOTTOM = 30 FT.

UNIT WEIGHT OF BACKFILL = 116 PCF

APPLIED LOAD DUE TO PROPOSED STRUCTURE = 21.6 KIPS
APPLIED STRESS DUE TO PROPOSED STRUCTURE = 1.2 TSF
APPLIED STRESS DUE TO BACKFILL = .116 TSF
TOTAL APPLIED STRESS = 1.316 TSF
NET APPLIED STRESS = 1.2 TSF

SETTLEMENT OCCURS FROM 2 FT BELOW ORIGINAL GROUND SURFACE
ESTIMATED SETTLEMENT OF SEGMENT NO. 1 = .228 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 2 = .168 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 3 = 8.700001E-02 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 4 = .042 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 5 = .042 IN.

  TOTAL ESTIMATED SETTLEMENT = .5670001 IN.

DISTRIBUTION OF VERTICAL EARTH PRESSURE

  D : DEPTH DOWN FROM ORIGINAL GROUND SURFACE (FT.)
  VP : VERTICAL EARTH PRESSURE BEFORE FOUNDATION EXCAVATION (TSF)
  IND : COEFFICIENT OF APPLIED STRESS DISTRIBUTION
  IVP : INDUCED VERTICAL EARTH PRESSURE (TSF)
  FVP : VERTICAL EARTH PRESSURE AFTER CONSTRUCTION (TSF)

D	VP	IND	IVP	FVP	SEGMENT NO.
2	.116	1	1.316	1.316	SEGMENT 1
3	.174	.829323	1.091389	1.149389	SEGMENT 1
4	.232	.5550288	.730418	.8464179	SEGMENT 2
5	.29	.3391858	.4463685	.6203685	SEGMENT 3
6	.348	.2187886	.2879259	.5199259	SEGMENT 4
7	.406	.1500931	.1975226	.4875226	SEGMENT 5
8	.464	.1084381	.1427045	.4907045	SEGMENT 5

```
***********************
* SETTLEMENT ANALYSIS *
***********************
```

PROJECT NAME : CYG-19-8863 Square Ftg 4' x 4' / Q = 2800 psf / D = 2'

TYPE OF FOOTING : SQUARE FOOTING

GEOMETRY OF FOOTING :
  DEFINE DEPTH OF ORIGINAL GROUND SURFACE AS 0 FT
  RELATIVE DEPTH TO BOTTOM OF FOOTING = 2 FT.
  THICKNESS OF BACKFILL ABOVE BOTTOM OF FOOTING = 2 FT.
  THICKNESS OF COMPACTED FILL BELOW FOOTING = 0 FT.
  SIDE LENGTH OF FOOTING = 4 FT.

SOIL LAYER NO. 1
  UNIT WEIGHT = 116 PCF
  DEPTH OF LAYER BOTTOM = 30 FT.

UNIT WEIGHT OF BACKFILL = 116 PCF

APPLIED LOAD DUE TO PROPOSED STRUCTURE = 44.8 KIPS
APPLIED STRESS DUE TO PROPOSED STRUCTURE = 1.4 TSF
APPLIED STRESS DUE TO BACKFILL = .116 TSF
TOTAL APPLIED STRESS = 1.516 TSF
NET APPLIED STRESS = 1.4 TSF

SETTLEMENT OCCURS FROM 2 FT BELOW ORIGINAL GROUND SURFACE
ESTIMATED SETTLEMENT OF SEGMENT NO. 1 = .249 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 2 = .207 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 3 = .156 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 4 = .108 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 5 = .132 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 6 = 6.120003E-02 IN.

  TOTAL ESTIMATED SETTLEMENT = .9132 IN.

DISTRIBUTION OF VERTICAL EARTH PRESSURE

  D : DEPTH DOWN FROM ORIGINAL GROUND SURFACE (FT.)
  VP : VERTICAL EARTH PRESSURE BEFORE FOUNDATION EXCAVATION (TSF)
  IND : COEFFICIENT OF APPLIED STRESS DISTRIBUTION
  IVP : INDUCED VERTICAL EARTH PRESSURE (TSF)
  FVP : VERTICAL EARTH PRESSURE AFTER CONSTRUCTION (TSF)

D	VP	IND	IVP	FVP	SEGMENT NO.
2	.116	1	1.516	1.516	SEGMENT 1
3	.174	.9105572	1.380405	1.438405	SEGMENT 1
4	.232	.7080709	1.073435	1.189436	SEGMENT 2
5	.29	.4894731	.7420412	.9160412	SEGMENT 3
6	.348	.3391858	.5142057	.7462056	SEGMENT 4
7	.406	.2426672	.3678835	.6578835	SEGMENT 5
8	.464	.1799187	.2727568	.6207568	SEGMENT 5
9	.522	.1377679	.2088561	.614856	SEGMENT 6

10	.58	.1084381	.1643921	.6283921	SEGMENT 6

```
***********************
* SETTLEMENT ANALYSIS *
***********************
```

PROJECT NAME : CYG-19-8863 Square Ftg 5' x 5' / Q = 3200 psf / D = 2'

TYPE OF FOOTING : SQUARE FOOTING

GEOMETRY OF FOOTING :
  DEFINE DEPTH OF ORIGINAL GROUND SURFACE AS 0 FT
  RELATIVE DEPTH TO BOTTOM OF FOOTING = 2 FT.
  THICKNESS OF BACKFILL ABOVE BOTTOM OF FOOTING = 2 FT.
  THICKNESS OF COMPACTED FILL BELOW FOOTING = 0 FT.
  SIDE LENGTH OF FOOTING = 5 FT.

SOIL LAYER NO. 1
  UNIT WEIGHT = 116 PCF
  DEPTH OF LAYER BOTTOM = 30 FT.

UNIT WEIGHT OF BACKFILL = 116 PCF

APPLIED LOAD DUE TO PROPOSED STRUCTURE = 80 KIPS
APPLIED STRESS DUE TO PROPOSED STRUCTURE = 1.6 TSF
APPLIED STRESS DUE TO BACKFILL = .116 TSF
TOTAL APPLIED STRESS = 1.716 TSF
NET APPLIED STRESS = 1.6 TSF

SETTLEMENT OCCURS FROM 2 FT BELOW ORIGINAL GROUND SURFACE
ESTIMATED SETTLEMENT OF SEGMENT NO. 1 = .2604 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 2 = .2214 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 3 = .1872 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 4 = .1632 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 5 = .228 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 6 = .108 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 7 = 5.999995E-02 IN.

  TOTAL ESTIMATED SETTLEMENT = 1.2282 IN.

DISTRIBUTION OF VERTICAL EARTH PRESSURE

  D : DEPTH DOWN FROM ORIGINAL GROUND SURFACE (FT.)
  VP : VERTICAL EARTH PRESSURE BEFORE FOUNDATION EXCAVATION (TSF)
  IND : COEFFICIENT OF APPLIED STRESS DISTRIBUTION
  IVP : INDUCED VERTICAL EARTH PRESSURE (TSF)
  FVP : VERTICAL EARTH PRESSURE AFTER CONSTRUCTION (TSF)

D	VP	IND	IVP	FVP	SEGMENT NO.
2	.116	1	1.716	1.716	SEGMENT 1
3	.174	.9487737	1.628096	1.686096	SEGMENT 1
4	.232	.7562166	1.297668	1.413668	SEGMENT 2
5	.29	.6131729	1.052205	1.226205	SEGMENT 3
6	.348	.4540456	.7791421	1.011142	SEGMENT 4
7	.406	.3391858	.5820428	.8720428	SEGMENT 5
8	.464	.2587242	.4439708	.7919708	SEGMENT 5

9	.522	.2019574	.3465589	.7525589	SEGMENT 6
10	.58	.1611158	.2764747	.7404746	SEGMENT 6
11	.6379999	.1310576	.2248948	.7468947	SEGMENT 7
12	.6959999	.1084381	.1860797	.7660796	SEGMENT 7

```
***********************
* SETTLEMENT ANALYSIS *
***********************
```

PROJECT NAME : CYG-19-8863 Square Ftg 2' x 2' / Q = 3200 psf / D = 2'

TYPE OF FOOTING : SQUARE FOOTING

GEOMETRY OF FOOTING :
DEFINE DEPTH OF ORIGINAL GROUND SURFACE AS 0 FT
RELATIVE DEPTH TO BOTTOM OF FOOTING = 2 FT.
THICKNESS OF BACKFILL ABOVE BOTTOM OF FOOTING = 2 FT.
THICKNESS OF COMPACTED FILL BELOW FOOTING = 0 FT.
SIDE LENGTH OF FOOTING = 2 FT.

SOIL LAYER NO. 1
UNIT WEIGHT = 116 PCF
DEPTH OF LAYER BOTTOM = 30 FT.

UNIT WEIGHT OF BACKFILL = 116 PCF

APPLIED LOAD DUE TO PROPOSED STRUCTURE = 12.8 KIPS
APPLIED STRESS DUE TO PROPOSED STRUCTURE = 1.6 TSF
APPLIED STRESS DUE TO BACKFILL = .116 TSF
TOTAL APPLIED STRESS = 1.716 TSF
NET APPLIED STRESS = 1.6 TSF

SETTLEMENT OCCURS FROM 2 FT BELOW ORIGINAL GROUND SURFACE
ESTIMATED SETTLEMENT OF SEGMENT NO. 1 = .246 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 2 = .153 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 3 = .06 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 4 = 2.700001E-02 IN.

TOTAL ESTIMATED SETTLEMENT = .4860001 IN.

DISTRIBUTION OF VERTICAL EARTH PRESSURE

D : DEPTH DOWN FROM ORIGINAL GROUND SURFACE (FT.)
VP : VERTICAL EARTH PRESSURE BEFORE FOUNDATION EXCAVATION (TSF)
IND : COEFFICIENT OF APPLIED STRESS DISTRIBUTION
IVP : INDUCED VERTICAL EARTH PRESSURE (TSF)
FVP : VERTICAL EARTH PRESSURE AFTER CONSTRUCTION (TSF)

D	VP	IND	IVP	FVP	SEGMENT NO.
2	.116	1	1.716	1.716	SEGMENT 1
3	.174	.7080709	1.21505	1.27305	SEGMENT 1
4	.232	.3391858	.5820428	.6980428	SEGMENT 2
5	.29	.1799187	.3087405	.4827405	SEGMENT 3
6	.348	.1084381	.1860797	.4180797	SEGMENT 4

```
**********************
* SETTLEMENT ANALYSIS *
**********************
```

PROJECT NAME : CYG-19-8863 Square Ftg 3' x 3' / Q = 3840 psf / D = 2'

TYPE OF FOOTING : SQUARE FOOTING

GEOMETRY OF FOOTING :
  DEFINE DEPTH OF ORIGINAL GROUND SURFACE AS 0 FT
  RELATIVE DEPTH TO BOTTOM OF FOOTING = 2 FT.
  THICKNESS OF BACKFILL ABOVE BOTTOM OF FOOTING = 2 FT.
  THICKNESS OF COMPACTED FILL BELOW FOOTING = 0 FT.
  SIDE LENGTH OF FOOTING = 3 FT.

SOIL LAYER NO. 1
  UNIT WEIGHT = 116 PCF
  DEPTH OF LAYER BOTTOM = 30 FT.

UNIT WEIGHT OF BACKFILL = 116 PCF

APPLIED LOAD DUE TO PROPOSED STRUCTURE = 34.56 KIPS
APPLIED STRESS DUE TO PROPOSED STRUCTURE = 1.92 TSF
APPLIED STRESS DUE TO BACKFILL = .116 TSF
TOTAL APPLIED STRESS = 2.036 TSF
NET APPLIED STRESS = 1.92 TSF

SETTLEMENT OCCURS FROM 2 FT BELOW ORIGINAL GROUND SURFACE
ESTIMATED SETTLEMENT OF SEGMENT NO. 1 = .27 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 2 = .2184 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 3 = .1554 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 4 = 9.299999E-02 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 5 = 8.399999E-02 IN.

TOTAL ESTIMATED SETTLEMENT = .8208 IN.

DISTRIBUTION OF VERTICAL EARTH PRESSURE

D : DEPTH DOWN FROM ORIGINAL GROUND SURFACE (FT.)
VP : VERTICAL EARTH PRESSURE BEFORE FOUNDATION EXCAVATION (TSF)
IND : COEFFICIENT OF APPLIED STRESS DISTRIBUTION
IVP : INDUCED VERTICAL EARTH PRESSURE (TSF)
FVP : VERTICAL EARTH PRESSURE AFTER CONSTRUCTION (TSF)

D	VP	IND	IVP	FVP	SEGMENT NO.
2	.116	1	2.036	2.036	SEGMENT 1
3	.174	.829323	1.688502	1.746502	SEGMENT 1
4	.232	.5550288	1.130039	1.246039	SEGMENT 2
5	.29	.3391858	.6905822	.8645822	SEGMENT 3
6	.348	.2187886	.4454537	.6774536	SEGMENT 4
7	.406	.1500931	.3055896	.5955896	SEGMENT 5
8	.464	.1084381	.2207799	.5687799	SEGMENT 5

```
************************
* SETTLEMENT ANALYSIS *
************************
```

PROJECT NAME : CYG-19-8863 Square Ftg 4' x 4' / Q = 4480 psf / D = 2'

TYPE OF FOOTING : SQUARE FOOTING

GEOMETRY OF FOOTING :
DEFINE DEPTH OF ORIGINAL GROUND SURFACE AS 0 FT
RELATIVE DEPTH TO BOTTOM OF FOOTING = 2 FT.
THICKNESS OF BACKFILL ABOVE BOTTOM OF FOOTING = 2 FT.
THICKNESS OF COMPACTED FILL BELOW FOOTING = 0 FT.
SIDE LENGTH OF FOOTING = 4 FT.

SOIL LAYER NO. 1
UNIT WEIGHT = 116 PCF
DEPTH OF LAYER BOTTOM = 30 FT.

UNIT WEIGHT OF BACKFILL = 116 PCF

APPLIED LOAD DUE TO PROPOSED STRUCTURE = 71.68 KIPS
APPLIED STRESS DUE TO PROPOSED STRUCTURE = 2.24 TSF
APPLIED STRESS DUE TO BACKFILL = .116 TSF
TOTAL APPLIED STRESS = 2.356 TSF
NET APPLIED STRESS = 2.24 TSF

SETTLEMENT OCCURS FROM 2 FT BELOW ORIGINAL GROUND SURFACE
ESTIMATED SETTLEMENT OF SEGMENT NO. 1 = .2856 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 2 = .249 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 3 = .2046 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 4 = .1686 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 5 = .222 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 6 = 9.000003E-02 IN.

TOTAL ESTIMATED SETTLEMENT = 1.2198 IN.

DISTRIBUTION OF VERTICAL EARTH PRESSURE

D : DEPTH DOWN FROM ORIGINAL GROUND SURFACE (FT.)
VP : VERTICAL EARTH PRESSURE BEFORE FOUNDATION EXCAVATION (TSF)
IND : COEFFICIENT OF APPLIED STRESS DISTRIBUTION
IVP : INDUCED VERTICAL EARTH PRESSURE (TSF)
FVP : VERTICAL EARTH PRESSURE AFTER CONSTRUCTION (TSF)

D	VP	IND	IVP	FVP	SEGMENT NO.
2	.116	1	2.356	2.356	SEGMENT 1
3	.174	.9105572	2.145273	2.203273	SEGMENT 1
4	.232	.7080709	1.668215	1.784215	SEGMENT 2
5	.29	.4894731	1.153199	1.327199	SEGMENT 3
6	.348	.3391858	.7991217	1.031122	SEGMENT 4
7	.406	.2426672	.5717239	.8617238	SEGMENT 5
8	.464	.1799187	.4238885	.7718885	SEGMENT 5
9	.522	.1377679	.3245811	.730581	SEGMENT 6

10	.58	.1084381	.2554801	.7194801	SEGMENT 6

```
***********************
* SETTLEMENT ANALYSIS *
***********************
```

PROJECT NAME : CYG-19-8863 Square Ftg 5' x 5' / Q = 5120 psf / D = 2'

TYPE OF FOOTING : SQUARE FOOTING

GEOMETRY OF FOOTING :
  DEFINE DEPTH OF ORIGINAL GROUND SURFACE AS 0 FT
  RELATIVE DEPTH TO BOTTOM OF FOOTING = 2 FT.
  THICKNESS OF BACKFILL ABOVE BOTTOM OF FOOTING = 2 FT.
  THICKNESS OF COMPACTED FILL BELOW FOOTING = 0 FT.
  SIDE LENGTH OF FOOTING = 5 FT.

SOIL LAYER NO. 1
  UNIT WEIGHT = 116 PCF
  DEPTH OF LAYER BOTTOM = 30 FT.

UNIT WEIGHT OF BACKFILL = 116 PCF

APPLIED LOAD DUE TO PROPOSED STRUCTURE = 128 KIPS
APPLIED STRESS DUE TO PROPOSED STRUCTURE = 2.56 TSF
APPLIED STRESS DUE TO BACKFILL = .116 TSF
TOTAL APPLIED STRESS = 2.676 TSF
NET APPLIED STRESS = 2.56 TSF

SETTLEMENT OCCURS FROM 2 FT BELOW ORIGINAL GROUND SURFACE
ESTIMATED SETTLEMENT OF SEGMENT NO. 1 = .303 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 2 = .261 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 3 = .231 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 4 = .204 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 5 = .33 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 6 = .216 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 7 = .1439999 IN.

TOTAL ESTIMATED SETTLEMENT = 1.689 IN.

DISTRIBUTION OF VERTICAL EARTH PRESSURE

  D : DEPTH DOWN FROM ORIGINAL GROUND SURFACE (FT.)
  VP : VERTICAL EARTH PRESSURE BEFORE FOUNDATION EXCAVATION (TSF)
  IND : COEFFICIENT OF APPLIED STRESS DISTRIBUTION
  IVP : INDUCED VERTICAL EARTH PRESSURE (TSF)
  FVP : VERTICAL EARTH PRESSURE AFTER CONSTRUCTION (TSF)

D	VP	IND	IVP	FVP	SEGMENT NO.
2	.116	1	2.676	2.676	SEGMENT 1
3	.174	.9487737	2.538918	2.596918	SEGMENT 1
4	.232	.7562166	2.023635	2.139635	SEGMENT 2
5	.29	.6131729	1.640851	1.814851	SEGMENT 3
6	.348	.4540456	1.215026	1.447026	SEGMENT 4
7	.406	.3391858	.9076611	1.197661	SEGMENT 5
8	.464	.2587242	.692346	1.040346	SEGMENT 5

9	.522	.2019574	.540438	.946438	SEGMENT 6
10	.58	.1611158	.4311458	.8951457	SEGMENT 6
11	.6379999	.1310576	.35071	.8727099	SEGMENT 7
12	.6959999	.1084381	.2901803	.8701801	SEGMENT 7

```
***********************
* SETTLEMENT ANALYSIS *
***********************
```

PROJECT NAME : CYG-19-8863 Continuous Ftg W = 1.5' / Q = 2000 psf / D = 2'

TYPE OF FOOTING : CONTINUOUS FOOTING

GEOMETRY OF FOOTING :
  DEFINE DEPTH OF ORIGINAL GROUND SURFACE AS 0 FT
  RELATIVE DEPTH TO BOTTOM OF FOOTING = 2 FT.
  THICKNESS OF BACKFILL ABOVE BOTTOM OF FOOTING = 2 FT.
  THICKNESS OF COMPACTED FILL BELOW FOOTING = 0 FT.
  WIDTH OF FOOTING = 1.5 FT.

SOIL LAYER NO. 1
  UNIT WEIGHT = 116 PCF
  DEPTH OF LAYER BOTTOM = 30 FT.

UNIT WEIGHT OF BACKFILL = 116 PCF

APPLIED STRESS DUE TO PROPOSED STRUCTURE = 1 TSF
APPLIED STRESS DUE TO BACKFILL = .116 TSF
TOTAL APPLIED STRESS = 1.116 TSF
NET APPLIED STRESS = .9999999 TSF

SETTLEMENT OCCURS FROM 2 FT BELOW ORIGINAL GROUND SURFACE
ESTIMATED SETTLEMENT OF SEGMENT NO. 1 = .186 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 2 = 9.900001E-02 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 3 = 4.800001E-02 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 4 = .03 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 5 = 2.400002E-02 IN.

TOTAL ESTIMATED SETTLEMENT = .387 IN.

DISTRIBUTION OF VERTICAL EARTH PRESSURE

D : DEPTH DOWN FROM ORIGINAL GROUND SURFACE (FT.)
VP : VERTICAL EARTH PRESSURE BEFORE FOUNDATION EXCAVATION (TSF)
IND : COEFFICIENT OF APPLIED STRESS DISTRIBUTION
IVP : INDUCED VERTICAL EARTH PRESSURE (TSF)
FVP : VERTICAL EARTH PRESSURE AFTER CONSTRUCTION (TSF)

D	VP	IND	IVP	FVP	SEGMENT NO.
2	.116	1	1.116	1.116	SEGMENT 1
3	.174	.669433	.7470871	.8050871	SEGMENT 1
4	.232	.4481405	.5001247	.6161247	SEGMENT 2
5	.29	.3	.3348	.5088	SEGMENT 3
6	.348	.2008299	.2241261	.4561261	SEGMENT 4
7	.406	.1344421	.1500374	.4400375	SEGMENT 5
8	.464	9.000001E-02	.10044	.44844	SEGMENT 5

```
***********************
* SETTLEMENT ANALYSIS *
***********************
```

PROJECT NAME : CYG-19-8863 Continuous Ftg W = 2' / Q = 2200 psf / D = 2'

TYPE OF FOOTING : CONTINUOUS FOOTING

GEOMETRY OF FOOTING :
DEFINE DEPTH OF ORIGINAL GROUND SURFACE AS 0 FT
RELATIVE DEPTH TO BOTTOM OF FOOTING = 2 FT.
THICKNESS OF BACKFILL ABOVE BOTTOM OF FOOTING = 2 FT.
THICKNESS OF COMPACTED FILL BELOW FOOTING = 0 FT.
WIDTH OF FOOTING = 2 FT.

SOIL LAYER NO. 1
UNIT WEIGHT = 116 PCF
DEPTH OF LAYER BOTTOM = 30 FT.

UNIT WEIGHT OF BACKFILL = 116 PCF

APPLIED STRESS DUE TO PROPOSED STRUCTURE = 1.1 TSF
APPLIED STRESS DUE TO BACKFILL = .116 TSF
TOTAL APPLIED STRESS = 1.216 TSF
NET APPLIED STRESS = 1.1 TSF

SETTLEMENT OCCURS FROM 2 FT BELOW ORIGINAL GROUND SURFACE
ESTIMATED SETTLEMENT OF SEGMENT NO. 1 = .2082 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 2 = .144 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 3 = 8.700001E-02 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 4 = .051 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 5 = 5.400002E-02 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 6 = 1.200002E-02 IN.

TOTAL ESTIMATED SETTLEMENT = .5562 IN.

DISTRIBUTION OF VERTICAL EARTH PRESSURE

D : DEPTH DOWN FROM ORIGINAL GROUND SURFACE (FT.)
VP : VERTICAL EARTH PRESSURE BEFORE FOUNDATION EXCAVATION (TSF)
IND : COEFFICIENT OF APPLIED STRESS DISTRIBUTION
IVP : INDUCED VERTICAL EARTH PRESSURE (TSF)
FVP : VERTICAL EARTH PRESSURE AFTER CONSTRUCTION (TSF)

D	VP	IND	IVP	FVP	SEGMENT NO.
2	.116	1	1.216	1.216	SEGMENT 1
3	.174	.7400828	.8999408	.9579408	SEGMENT 1
4	.232	.5477226	.6660307	.7820307	SEGMENT 2
5	.29	.4053601	.4929179	.6669179	SEGMENT 3
6	.348	.3	.3648	.5968	SEGMENT 4
7	.406	.2220249	.2699823	.5599823	SEGMENT 5
8	.464	.1643168	.1998092	.5478092	SEGMENT 5
9	.522	.121608	.1478754	.5538753	SEGMENT 6
10	.58	9.000001E-02	.10944	.57344	SEGMENT 6

```
***********************
* SETTLEMENT ANALYSIS *
***********************
```

PROJECT NAME : CYG-19-8863 Continuous Ftg W = 3' / Q = 2600 psf / D = 2'

TYPE OF FOOTING : CONTINUOUS FOOTING

GEOMETRY OF FOOTING :
DEFINE DEPTH OF ORIGINAL GROUND SURFACE AS 0 FT
RELATIVE DEPTH TO BOTTOM OF FOOTING = 2 FT.
THICKNESS OF BACKFILL ABOVE BOTTOM OF FOOTING = 2 FT.
THICKNESS OF COMPACTED FILL BELOW FOOTING = 0 FT.
WIDTH OF FOOTING = 3 FT.

SOIL LAYER NO. 1
UNIT WEIGHT = 116 PCF
DEPTH OF LAYER BOTTOM = 30 FT.

UNIT WEIGHT OF BACKFILL = 116 PCF

APPLIED STRESS DUE TO PROPOSED STRUCTURE = 1.3 TSF
APPLIED STRESS DUE TO BACKFILL = .116 TSF
TOTAL APPLIED STRESS = 1.416 TSF
NET APPLIED STRESS = 1.3 TSF

SETTLEMENT OCCURS FROM 2 FT BELOW ORIGINAL GROUND SURFACE
ESTIMATED SETTLEMENT OF SEGMENT NO. 1 = .2322 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 2 = .1902 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 3 = .15 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 4 = .123 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 5 = .18 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 6 = 9.600002E-02 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 7 = 6.239999E-02 IN.

TOTAL ESTIMATED SETTLEMENT = 1.0338 IN.

DISTRIBUTION OF VERTICAL EARTH PRESSURE

D : DEPTH DOWN FROM ORIGINAL GROUND SURFACE (FT.)
VP : VERTICAL EARTH PRESSURE BEFORE FOUNDATION EXCAVATION (TSF)
IND : COEFFICIENT OF APPLIED STRESS DISTRIBUTION
IVP : INDUCED VERTICAL EARTH PRESSURE (TSF)
FVP : VERTICAL EARTH PRESSURE AFTER CONSTRUCTION (TSF)

D	VP	IND	IVP	FVP	SEGMENT NO.
2	.116	1	1.416	1.416	SEGMENT 1
3	.174	.8181888	1.158555	1.216555	SEGMENT 1
4	.232	.669433	.947917	1.063917	SEGMENT 2
5	.29	.5477226	.7755751	.9495751	SEGMENT 3
6	.348	.4481405	.6345668	.8665668	SEGMENT 4
7	.406	.3666635	.5191955	.8091955	SEGMENT 5
8	.464	.3	.4248	.7728	SEGMENT 5
9	.522	.2454566	.3475666	.7535666	SEGMENT 6

10	.58	.2008299	.2843751	.748375	SEGMENT 6
11	.6379999	.1643168	.2326726	.7546725	SEGMENT 7
12	.6959999	.1344421	.1903701	.77037	SEGMENT 7
13	.7539998	.1099991	.1557587	.7937585	SEGMENT 7
14	.8119998	9.000001E-02	.12744	.8234398	SEGMENT 7

```
***********************
* SETTLEMENT ANALYSIS *
***********************
```

PROJECT NAME : CYG-19-8863 Continuous Ftg W = 4' / Q = 3000 psf / D = 2'

TYPE OF FOOTING : CONTINUOUS FOOTING

GEOMETRY OF FOOTING :
  DEFINE DEPTH OF ORIGINAL GROUND SURFACE AS 0 FT
  RELATIVE DEPTH TO BOTTOM OF FOOTING = 2 FT.
  THICKNESS OF BACKFILL ABOVE BOTTOM OF FOOTING = 2 FT.
  THICKNESS OF COMPACTED FILL BELOW FOOTING = 0 FT.
  WIDTH OF FOOTING = 4 FT.

SOIL LAYER NO. 1
  UNIT WEIGHT = 116 PCF
  DEPTH OF LAYER BOTTOM = 30 FT.

UNIT WEIGHT OF BACKFILL = 116 PCF

APPLIED STRESS DUE TO PROPOSED STRUCTURE = 1.5 TSF
APPLIED STRESS DUE TO BACKFILL = .116 TSF
TOTAL APPLIED STRESS = 1.616 TSF
NET APPLIED STRESS = 1.5 TSF

SETTLEMENT OCCURS FROM 2 FT BELOW ORIGINAL GROUND SURFACE
ESTIMATED SETTLEMENT OF SEGMENT NO. 1 = .255 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 2 = .216 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 3 = .1836 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 4 = .1686 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 5 = .504 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 6 = .2759999 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 7 = .132 IN.

TOTAL ESTIMATED SETTLEMENT = 1.7352 IN.

DISTRIBUTION OF VERTICAL EARTH PRESSURE

D : DEPTH DOWN FROM ORIGINAL GROUND SURFACE (FT.)
VP : VERTICAL EARTH PRESSURE BEFORE FOUNDATION EXCAVATION (TSF)
IND : COEFFICIENT OF APPLIED STRESS DISTRIBUTION
IVP : INDUCED VERTICAL EARTH PRESSURE (TSF)
FVP : VERTICAL EARTH PRESSURE AFTER CONSTRUCTION (TSF)

D	VP	IND	IVP	FVP	SEGMENT NO.
2	.116	1	1.616	1.616	SEGMENT 1
3	.174	.8602806	1.390214	1.448214	SEGMENT 1
4	.232	.7400828	1.195974	1.311974	SEGMENT 2
5	.29	.636679	1.028873	1.202873	SEGMENT 3
6	.348	.5477226	.8851197	1.11712	SEGMENT 4
7	.406	.4711952	.7614513	1.051451	SEGMENT 5
8	.464	.4053601	.6550618	1.003062	SEGMENT 5
9	.522	.3487234	.563537	.9695369	SEGMENT 5

10	.58	.3	.4848	.9487999	SEGMENT 5
11	.6379999	.2580842	.4170641	.9390639	SEGMENT 6
12	.6959999	.2220249	.3587922	.9387921	SEGMENT 6
13	.7539998	.1910037	.308662	.9466618	SEGMENT 6
14	.8119998	.1643168	.2655359	.9615358	SEGMENT 6
15	.8699998	.1413585	.2284354	.9824352	SEGMENT 7
16	.9279998	.121608	.1965186	1.008518	SEGMENT 7
17	.9859997	.104617	.1690611	1.039061	SEGMENT 7
18	1.044	9.000001E-02	.14544	1.07344	SEGMENT 7

```
************************
* SETTLEMENT ANALYSIS *
************************
```

PROJECT NAME : CYG-19-8863 Continuous Ftg W = 1.5' / Q = 3200 psf / D = 2'

TYPE OF FOOTING : CONTINUOUS FOOTING

GEOMETRY OF FOOTING :
DEFINE DEPTH OF ORIGINAL GROUND SURFACE AS 0 FT
RELATIVE DEPTH TO BOTTOM OF FOOTING = 2 FT.
THICKNESS OF BACKFILL ABOVE BOTTOM OF FOOTING = 2 FT.
THICKNESS OF COMPACTED FILL BELOW FOOTING = 0 FT.
WIDTH OF FOOTING = 1.5 FT.

SOIL LAYER NO. 1
UNIT WEIGHT = 116 PCF
DEPTH OF LAYER BOTTOM = 30 FT.

UNIT WEIGHT OF BACKFILL = 116 PCF

APPLIED STRESS DUE TO PROPOSED STRUCTURE = 1.6 TSF
APPLIED STRESS DUE TO BACKFILL = .116 TSF
TOTAL APPLIED STRESS = 1.716 TSF
NET APPLIED STRESS = 1.6 TSF

SETTLEMENT OCCURS FROM 2 FT BELOW ORIGINAL GROUND SURFACE
ESTIMATED SETTLEMENT OF SEGMENT NO. 1 = .2436 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 2 = .1746 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 3 = .108 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 4 = .0582 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 5 = .0456 IN.

TOTAL ESTIMATED SETTLEMENT = .63 IN.

DISTRIBUTION OF VERTICAL EARTH PRESSURE

D : DEPTH DOWN FROM ORIGINAL GROUND SURFACE (FT.)
VP : VERTICAL EARTH PRESSURE BEFORE FOUNDATION EXCAVATION (TSF)
IND : COEFFICIENT OF APPLIED STRESS DISTRIBUTION
IVP : INDUCED VERTICAL EARTH PRESSURE (TSF)
FVP : VERTICAL EARTH PRESSURE AFTER CONSTRUCTION (TSF)

D	VP	IND	IVP	FVP	SEGMENT NO.
2	.116	1	1.716	1.716	SEGMENT 1
3	.174	.669433	1.148747	1.206747	SEGMENT 1
4	.232	.4481405	.7690091	.885009	SEGMENT 2
5	.29	.3	.5148001	.6888001	SEGMENT 3
6	.348	.2008299	.3446241	.5766241	SEGMENT 4
7	.406	.1344421	.2307027	.5207027	SEGMENT 5
8	.464	9.000001E-02	.15444	.5024401	SEGMENT 5

```
**********************
* SETTLEMENT ANALYSIS *
**********************
```

PROJECT NAME : CYG-19-8863 Continuous Ftg W = 2' / Q = 3520 psf / D = 2'

TYPE OF FOOTING : CONTINUOUS FOOTING

GEOMETRY OF FOOTING :
  DEFINE DEPTH OF ORIGINAL GROUND SURFACE AS 0 FT
  RELATIVE DEPTH TO BOTTOM OF FOOTING = 2 FT.
  THICKNESS OF BACKFILL ABOVE BOTTOM OF FOOTING = 2 FT.
  THICKNESS OF COMPACTED FILL BELOW FOOTING = 0 FT.
  WIDTH OF FOOTING = 2 FT.

SOIL LAYER NO. 1
  UNIT WEIGHT = 116 PCF
  DEPTH OF LAYER BOTTOM = 30 FT.

UNIT WEIGHT OF BACKFILL = 116 PCF

APPLIED STRESS DUE TO PROPOSED STRUCTURE = 1.76 TSF
APPLIED STRESS DUE TO BACKFILL = .116 TSF
TOTAL APPLIED STRESS = 1.876 TSF
NET APPLIED STRESS = 1.76 TSF

SETTLEMENT OCCURS FROM 2 FT BELOW ORIGINAL GROUND SURFACE
ESTIMATED SETTLEMENT OF SEGMENT NO. 1 = .258 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 2 = .21 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 3 = .159 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 4 = .114 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 5 = .1416 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 6 = 6.360001E-02 IN.

TOTAL ESTIMATED SETTLEMENT = .9461999 IN.

DISTRIBUTION OF VERTICAL EARTH PRESSURE

D : DEPTH DOWN FROM ORIGINAL GROUND SURFACE (FT.)
VP : VERTICAL EARTH PRESSURE BEFORE FOUNDATION EXCAVATION (TSF)
IND : COEFFICIENT OF APPLIED STRESS DISTRIBUTION
IVP : INDUCED VERTICAL EARTH PRESSURE (TSF)
FVP : VERTICAL EARTH PRESSURE AFTER CONSTRUCTION (TSF)

D	VP	IND	IVP	FVP	SEGMENT NO.
2	.116	1	1.876	1.876	SEGMENT 1
3	.174	.7400828	1.388395	1.446395	SEGMENT 1
4	.232	.5477226	1.027528	1.143528	SEGMENT 2
5	.29	.4053601	.7604555	.9344554	SEGMENT 3
6	.348	.3	.5628	.7948	SEGMENT 4
7	.406	.2220249	.4165186	.7065186	SEGMENT 5
8	.464	.1643168	.3082583	.6562583	SEGMENT 5
9	.522	.121608	.2281366	.6341366	SEGMENT 6
10	.58	9.000001E-02	.16884	.63284	SEGMENT 6

```
***********************
* SETTLEMENT ANALYSIS *
***********************
```

PROJECT NAME : CYG-19-8863 Continuous Ftg W = 3' / Q = 4160 psf / D = 2'

TYPE OF FOOTING : CONTINUOUS FOOTING

GEOMETRY OF FOOTING :
  DEFINE DEPTH OF ORIGINAL GROUND SURFACE AS 0 FT
  RELATIVE DEPTH TO BOTTOM OF FOOTING = 2 FT.
  THICKNESS OF BACKFILL ABOVE BOTTOM OF FOOTING = 2 FT.
  THICKNESS OF COMPACTED FILL BELOW FOOTING = 0 FT.
  WIDTH OF FOOTING = 3 FT.

SOIL LAYER NO. 1
  UNIT WEIGHT = 116 PCF
  DEPTH OF LAYER BOTTOM = 30 FT.

UNIT WEIGHT OF BACKFILL = 116 PCF

APPLIED STRESS DUE TO PROPOSED STRUCTURE = 2.08 TSF
APPLIED STRESS DUE TO BACKFILL = .116 TSF
TOTAL APPLIED STRESS = 2.196 TSF
NET APPLIED STRESS = 2.08 TSF

SETTLEMENT OCCURS FROM 2 FT BELOW ORIGINAL GROUND SURFACE
ESTIMATED SETTLEMENT OF SEGMENT NO. 1 = .276 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 2 = .2358 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 3 = .2028 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 4 = .1782 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 5 = .3024 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 6 = .2100001 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 7 = .192 IN.

TOTAL ESTIMATED SETTLEMENT = 1.5972 IN.

DISTRIBUTION OF VERTICAL EARTH PRESSURE

D : DEPTH DOWN FROM ORIGINAL GROUND SURFACE (FT.)
VP : VERTICAL EARTH PRESSURE BEFORE FOUNDATION EXCAVATION (TSF)
IND : COEFFICIENT OF APPLIED STRESS DISTRIBUTION
IVP : INDUCED VERTICAL EARTH PRESSURE (TSF)
FVP : VERTICAL EARTH PRESSURE AFTER CONSTRUCTION (TSF)

D	VP	IND	IVP	FVP	SEGMENT NO.
2	.116	1	2.196	2.196	SEGMENT 1
3	.174	.8181888	1.796743	1.854743	SEGMENT 1
4	.232	.669433	1.470075	1.586075	SEGMENT 2
5	.29	.5477226	1.202799	1.376799	SEGMENT 3
6	.348	.4481405	.9841163	1.216116	SEGMENT 4
7	.406	.3666635	.805193	1.095193	SEGMENT 5
8	.464	.3	.6588	1.0068	SEGMENT 5
9	.522	.2454566	.5390228	.9450228	SEGMENT 6

10	.58	.2008299	.4410224	.9050223	SEGMENT 6
11	.6379999	.1643168	.3608396	.8828395	SEGMENT 7
12	.6959999	.1344421	.2952349	.8752348	SEGMENT 7
13	.7539998	.1099991	.241558	.8795578	SEGMENT 7
14	.8119998	9.000001E-02	.19764	.8936399	SEGMENT 7

```
***********************
* SETTLEMENT ANALYSIS *
***********************
```

PROJECT NAME : CYG-19-8863 Continuous Ftg W = 4' / Q = 4800 psf / D = 2'

TYPE OF FOOTING : CONTINUOUS FOOTING

GEOMETRY OF FOOTING :
  DEFINE DEPTH OF ORIGINAL GROUND SURFACE AS 0 FT
  RELATIVE DEPTH TO BOTTOM OF FOOTING = 2 FT.
  THICKNESS OF BACKFILL ABOVE BOTTOM OF FOOTING = 2 FT.
  THICKNESS OF COMPACTED FILL BELOW FOOTING = 0 FT.
  WIDTH OF FOOTING = 4 FT.

SOIL LAYER NO. 1
  UNIT WEIGHT = 116 PCF
  DEPTH OF LAYER BOTTOM = 30 FT.

UNIT WEIGHT OF BACKFILL = 116 PCF

APPLIED STRESS DUE TO PROPOSED STRUCTURE = 2.4 TSF
APPLIED STRESS DUE TO BACKFILL = .116 TSF
TOTAL APPLIED STRESS = 2.516 TSF
NET APPLIED STRESS = 2.4 TSF

SETTLEMENT OCCURS FROM 2 FT BELOW ORIGINAL GROUND SURFACE
ESTIMATED SETTLEMENT OF SEGMENT NO. 1 = .288 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 2 = .252 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 3 = .2226 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 4 = .2052 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 5 = .7224001 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 6 = .5160001 IN.
ESTIMATED SETTLEMENT OF SEGMENT NO. 7 = .24 IN.

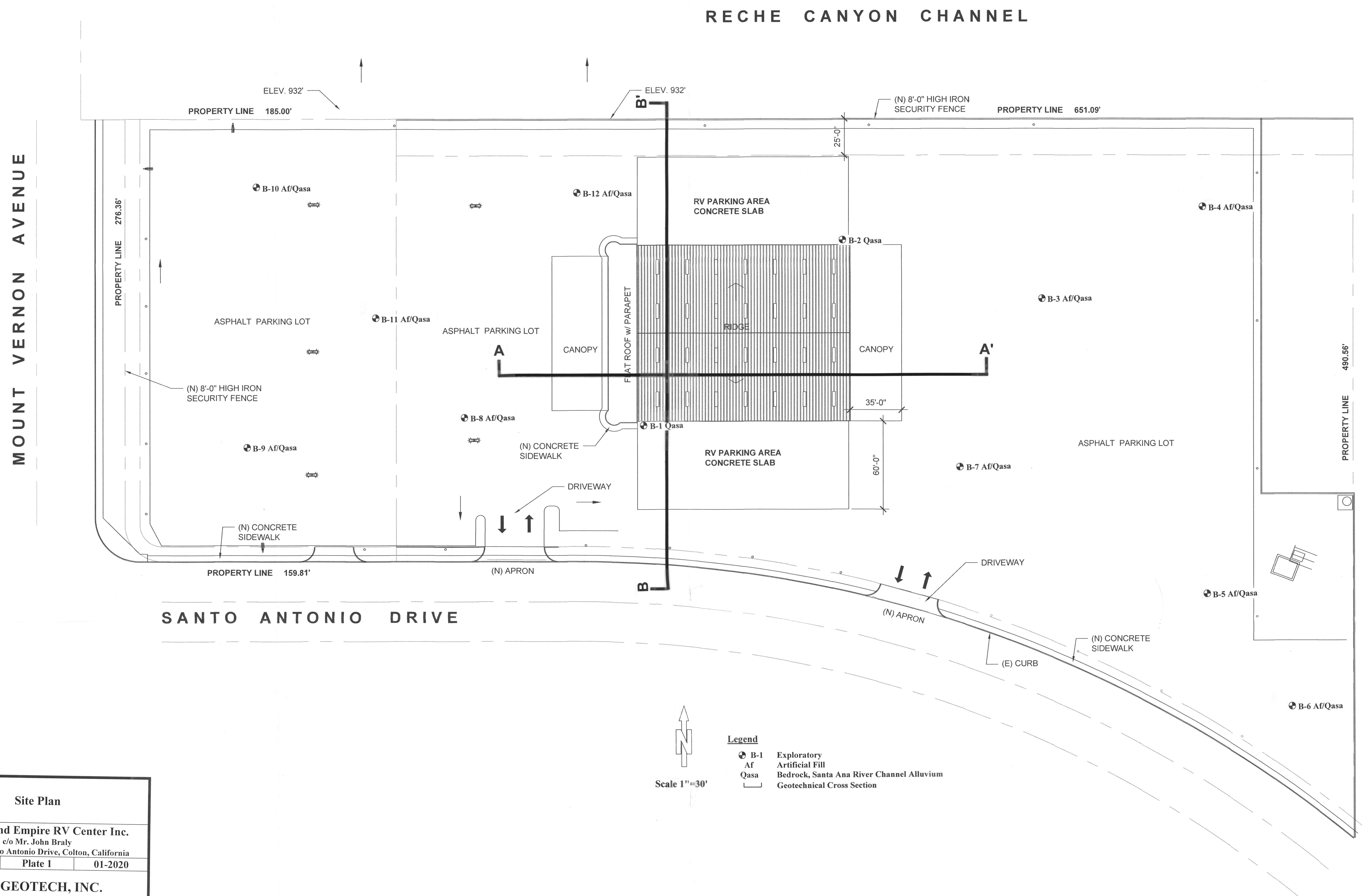
TOTAL ESTIMATED SETTLEMENT = 2.4462 IN.

DISTRIBUTION OF VERTICAL EARTH PRESSURE

  D : DEPTH DOWN FROM ORIGINAL GROUND SURFACE (FT.)
  VP : VERTICAL EARTH PRESSURE BEFORE FOUNDATION EXCAVATION (TSF)
  IND : COEFFICIENT OF APPLIED STRESS DISTRIBUTION
  IVP : INDUCED VERTICAL EARTH PRESSURE (TSF)
  FVP : VERTICAL EARTH PRESSURE AFTER CONSTRUCTION (TSF)

D	VP	IND	IVP	FVP	SEGMENT NO.
2	.116	1	2.516	2.516	SEGMENT 1
3	.174	.8602806	2.164466	2.222466	SEGMENT 1
4	.232	.7400828	1.862048	1.978049	SEGMENT 2
5	.29	.636679	1.601884	1.775884	SEGMENT 3
6	.348	.5477226	1.37807	1.61007	SEGMENT 4
7	.406	.4711952	1.185527	1.475527	SEGMENT 5
8	.464	.4053601	1.019886	1.367886	SEGMENT 5
9	.522	.3487234	.8773881	1.283388	SEGMENT 5

10	.58	.3	.7548	1.2188	SEGMENT 5
11	.6379999	.2580842	.6493399	1.17134	SEGMENT 6
12	.6959999	.2220249	.5586146	1.138614	SEGMENT 6
13	.7539998	.1910037	.4805653	1.118565	SEGMENT 6
14	.8119998	.1643168	.413421	1.109421	SEGMENT 6
15	.8699998	.1413585	.3556581	1.109658	SEGMENT 7
16	.9279998	.121608	.3059658	1.117966	SEGMENT 7
17	.9859997	.104617	.2632164	1.133216	SEGMENT 7
18	1.044	9.000001E-02	.22644	1.15444	SEGMENT 7



Site Plan

Giant Inland Empire RV Center Inc. c/o Mr. John Braly 1301 East Santo Antonio Drive, Colton, California		
CYG-19-8863	Plate 1	01-2020

C.Y. GEOTECH, INC.

Engineering Geology and Geotechnical Engineering