C. Y. GEOTECH, INC.

Engineering Geology and Geotechnical Engineering

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GEOTECHNICAL ENGINEERING INVESTIGATION

PROPOSED FIVE UNIT CONDOMINIUM
LOT S, BLOCK 136, SANTA MONICA TRACT
1327 EUCLID STREET
SANTA MONICA, CALIFORNIA

FOR

MR. VLADI TOMALEVSKI

MAY 10, 2006
PROJECT NO. CYG-06-4508

May 10, 2006

P.N. CYG-06-4508

Mr. Vladi Tomalevski
1514 17th Street
Santa Monica, California 90404

Subject: Geotechnical Engineering Investigation, Proposed Five Unit Condominium, Lot S, Block 136, Santa Monica Tract, 1327 Euclid Street, Santa Monica, California

Dear Vladi,

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

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

Conventional spread footings founded into competent older alluvium which is approximately 2 feet below the existing ground surface can be used to support the proposed condominium. Alternatively, deep foundation such as end bearing caissons or skin friction piles entirely founded into competent older alluvium can also be used to support the proposed condominium.

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

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

John T. Tsao
RCE 46886/CEG 1783

REGISTERED PROFESSIONAL ENGINEER
TZE-TZONG TSAO
No. C 046886
Exp. 06-30-07
CIVIL
STATE OF CALIFORNIA

REGISTERED GEOLOGIST
TZE-TZONG, TSAO
No. 1783
CERTIFIED ENGINEERING GEOLOGIST
EXP. 11-30-07
STATE OF CALIFORNIA

Encl: Appendix A, Field Exploration and Laboratory Testing

cc: (5) Addressee

GEOTECHNICAL ENGINEERING INVESTIGATION

Proposed Five Unit Condominium
Lot S, Block 136, Santa Monica Tract
1327 Euclid Street, Santa Monica, California

As requested, CYG has performed a geotechnical engineering investigation for the development of the subject project. The purpose of this investigation is 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 five unit condominium.

1.0 SCOPE OF WORK

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

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

b. Drilling, logging and sampling four (4) borings and to a maximum depth of 21 feet at locations as shown on Figure 2. Boring logs are presented in Appendix A.

c. Perform laboratory tests to determine the engineering properties of onsite earth materials. The results of laboratory tests are presented in Appendix A and summarized in Section 5.2.

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

e. Perform geotechnical engineering evaluations and analyses. Slope stability analyses were performed to calculate the equivalent fluid pressure for retaining wall design and to evaluate the stability of temporary excavations.

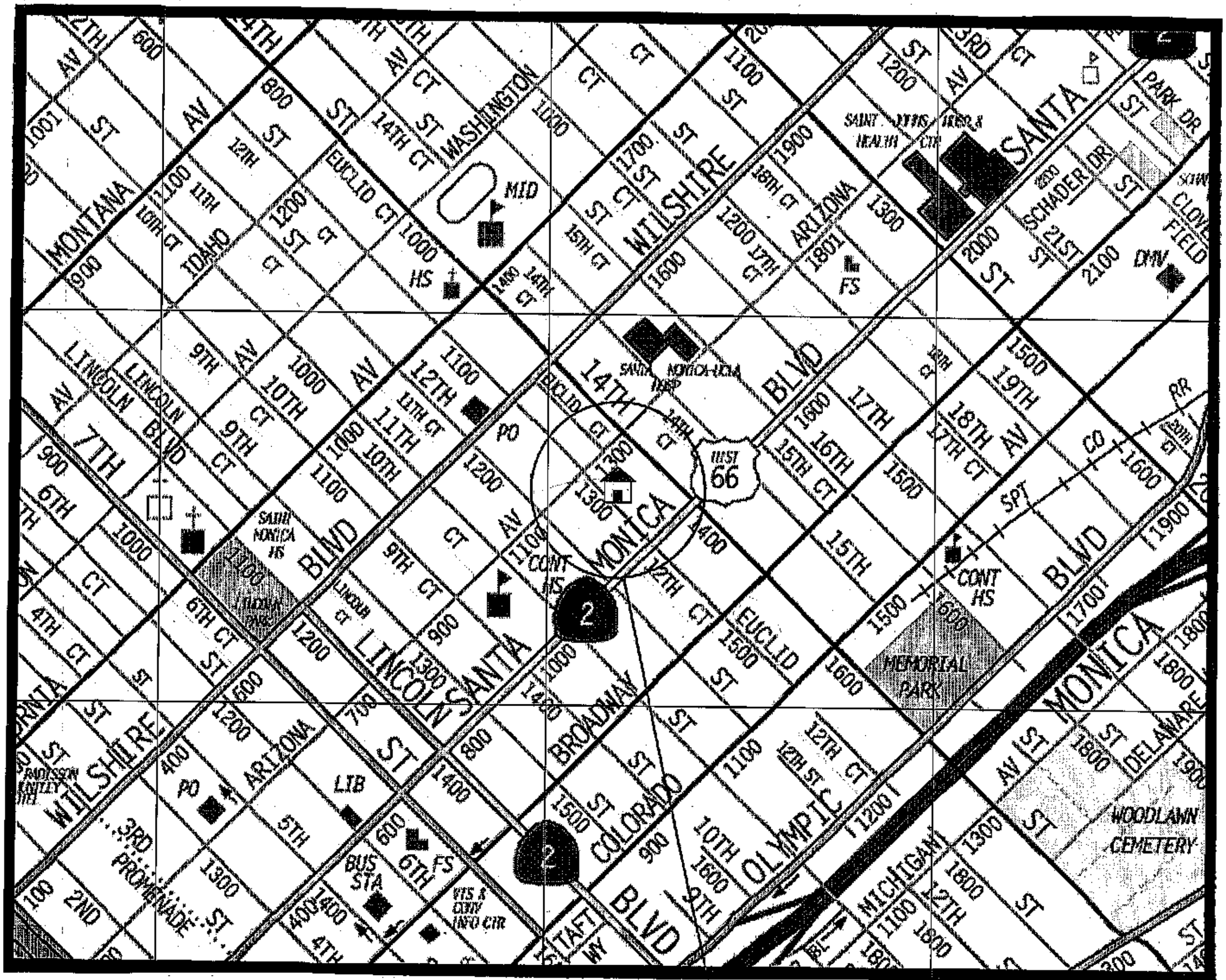
f. Prepare this report to present the findings and conclusions of this investigation and to provide recommendations for the design and construction of the proposed condominium.

2.0 SITE DESCRIPTION

The subject site is located at 1327 Euclid Street, Santa Monica, California. The legal description of the site is Lot S, Block 136, Santa Monica Tract. A site location map is shown on Figure 1. The site is bounded on the southwest by Euclid Street, on the northeast by Euclid Court, and on the northwest and southeast by neighboring residences. The site is rectangular-shaped and fairly level. The site is currently occupied by two single family residences, a detached garage and surrounding yard areas. A site plan showing the site, the property line, the existing structures and the proposed condominium is shown on Figure 2.

3.0 PROPOSED DEVELOPMENT

Information regarding the proposed development was provided by you and was used as a guide for the field exploration and report preparation. It is our understanding that the existing structures are to be demolished and a five unit condominium is to be built on the site. The condominium will be provided with a basement.

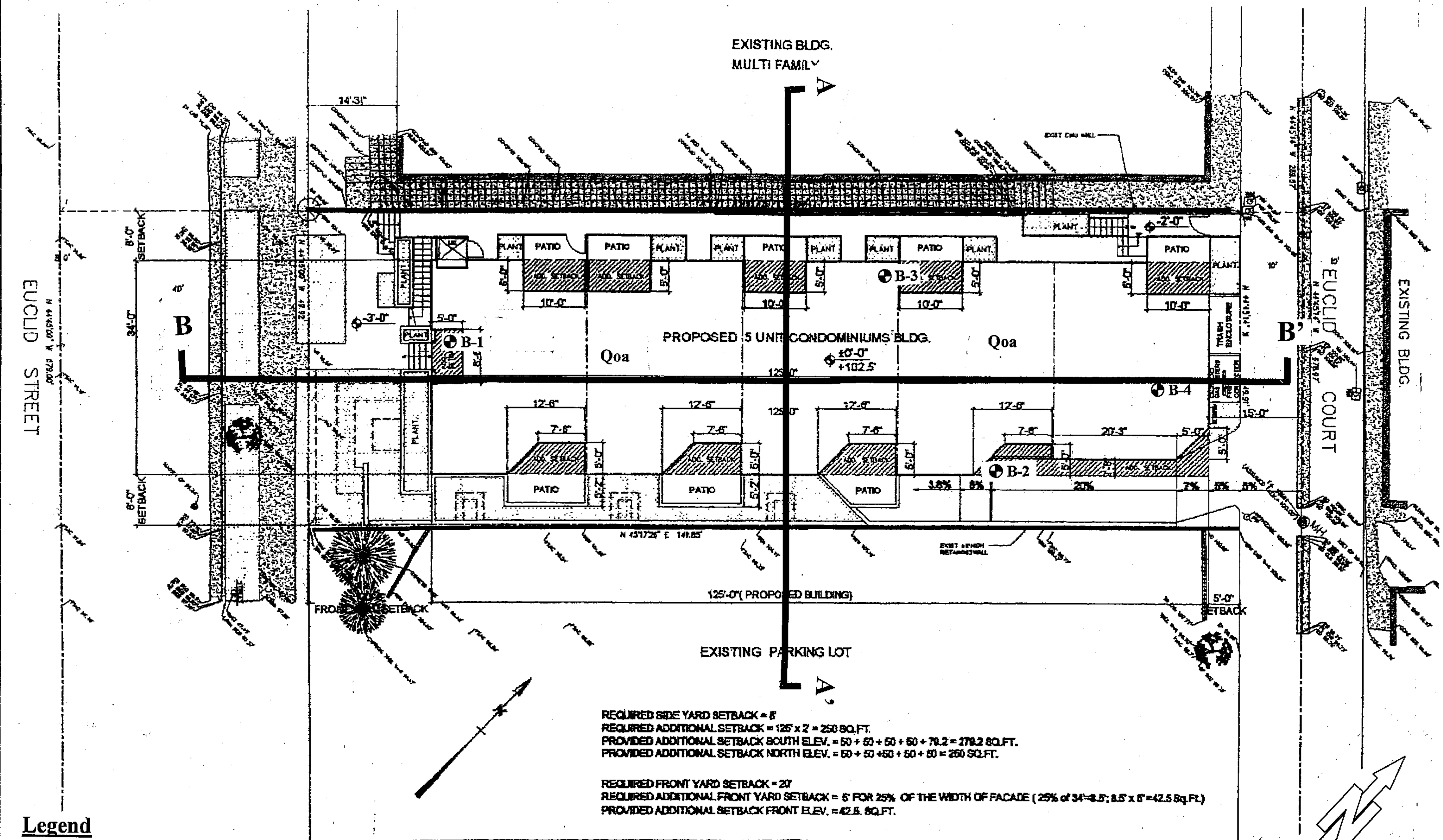


Subject Site

(Adopted from Thomas Brothers Maps, Los Angeles County, California, 1998. CD-ROM Version)

Scale 1"=1200'

C.Y. GEOTECH, INC. Engineering Geology and Geotechnical Engineering	**Site Location Map**
CYG-06-4508	**Figure 1**



Legend

B-1 Boring Locations
Qoa Older Alluvium
A A' Geologic Cross Section

Scale 1"= 16'

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Site Plan Showing Proposed Development

Figure 2

CYG-06-4508

A site plan showing the site, the property lines and the proposed development is shown on Figure 2. One architectural plan and one architectural cross section showing the garage level floor are shown on Figure 3. Two geotechnical cross sections showing the subsurface conditions of the site and immediate site vicinity are shown on Figure 4.

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

4.0 FIELD EXPLORATION AND LABORATORY TESTING

Field exploration was performed by one of our geologists on April 28, 2006 with the aid of hand laborers. Four (4) borings were drilled to a maximum depth of 21 feet at the locations as shown on Figure 2. Hand-operated augers and hand tools were used to drill borings. All borings were logged by the geologist and backfilled on the same day of field exploration. The boring logs are presented in Appendix A.

Ring samples of onsite earth materials were obtained by using a split-tube ring sampler. The ring samples 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 ring samples with the brass rings were further retained in plastic, close-fitting, moisture-tight containers. Bulk samples of onsite soils were collected for laboratory compaction test and expansion index test. Ring samples and bulk samples were delivered to CYG for laboratory tests.

Laboratory testing of onsite earth materials was performed after the review of field data and in consideration of the proposed development, the stability of the site and the probable foundation system to be used. The testing procedures of ASTM (American Society for Testing and Materials) Standards were followed in the laboratory testing. The following engineering properties of onsite earth materials are determined: 1) field density and field moisture content, 2) maximum dry density and optimum moisture content, 3) cohesion and friction angle, 4) compressibility and hydroconsolidation, and 5) expansion index. The results of laboratory tests are presented in Appendix A and summarized in Section 5.2.

5.0 EARTH MATERIAL

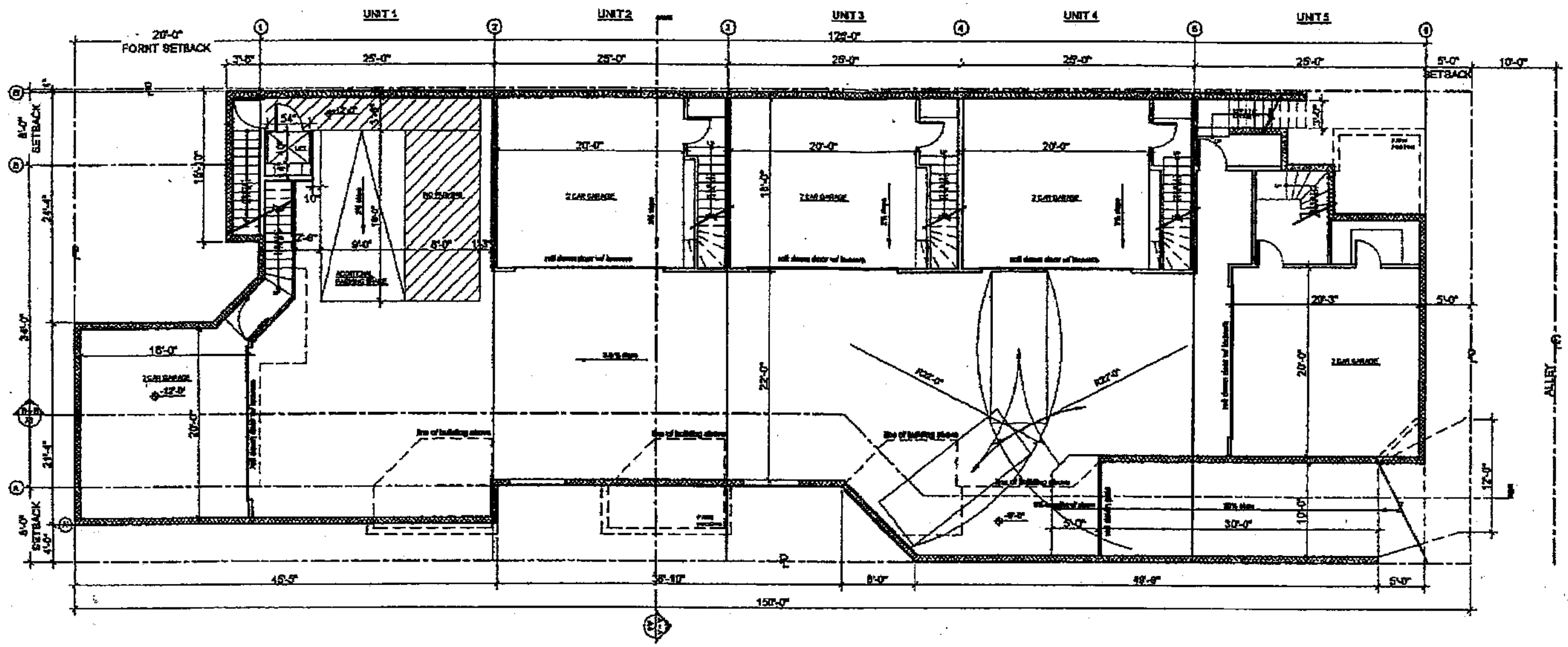
Earth materials encountered in the borings consisted of older alluvium. Descriptions of the earth materials encountered are shown on the boring logs enclosed in Appendix A. The engineering properties of onsite earth materials are presented in Appendix A and summarized in Section 5.2.

A geologic map showing the general geologic conditions of the site and site vicinity adopted from the Dibblee Geologic Map is shown on Figure 5. The alluvium encountered in the borings is consistent with the Dibblee Geologic Map.

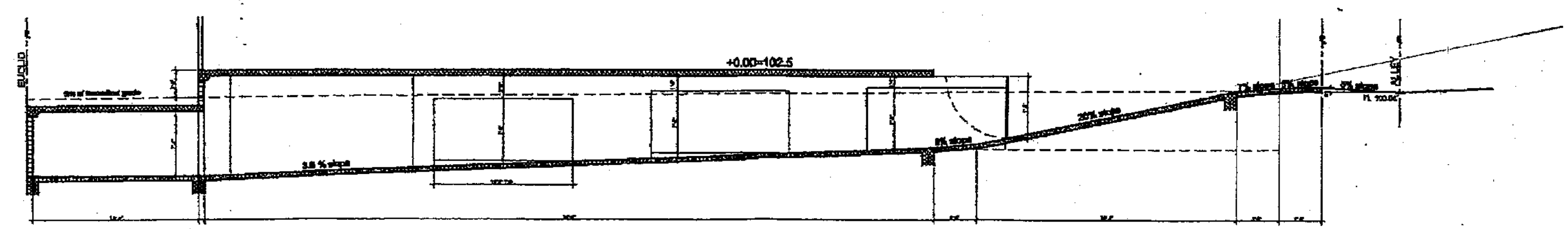
5.1 Older Alluvium (Qoa)

Older alluvium was observed from the ground surface to the depth explored in all borings. The older alluvium consisted of dark brown clayey sandy silt to brown gravelly sand in a moist to very moist and moderately dense to dense condition. Laboratory tests indicated a dry density of 92 to 124 pcf for the older alluvium.

The laboratory expansion index test indicated an expansion index of 15 for the tested older alluvium. A soil with an expansion index in the range of 0 to 20 is considered as a very low expansive soil. The laboratory consolidation tests indicated a hydorconsolidation potential of 5.3% for soil from boring B-2 @ 1 ft and a hydroconsolidation potential of 0 to 0.9% for soil deeper than 1 foot.



GARAGE LEVEL FLOOR PLAN



GARAGE SECTION *B-B*

Scale 1"= 16'

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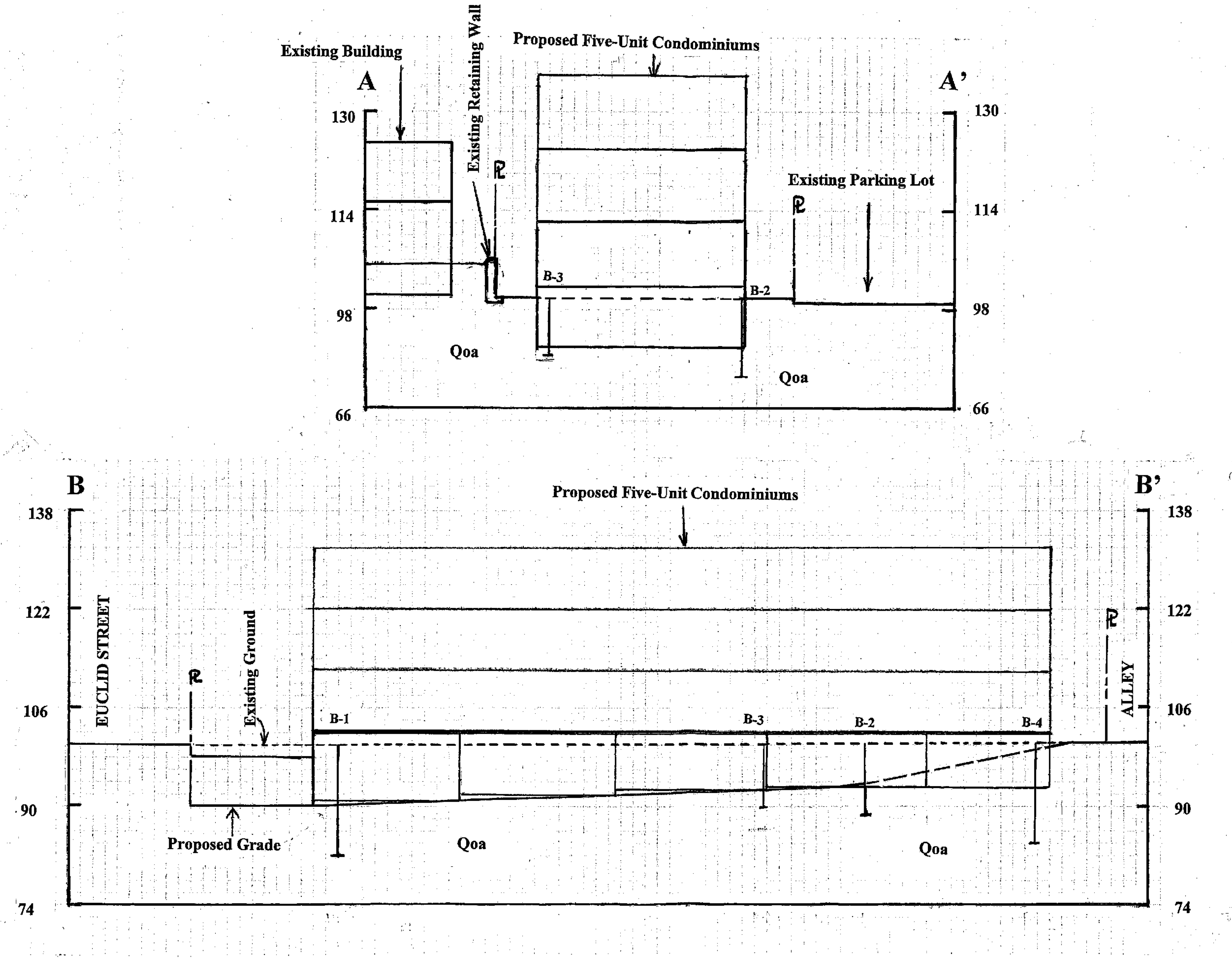
Architectural Garage Plan and Cross Section

Figure 3

CYG-06-4508



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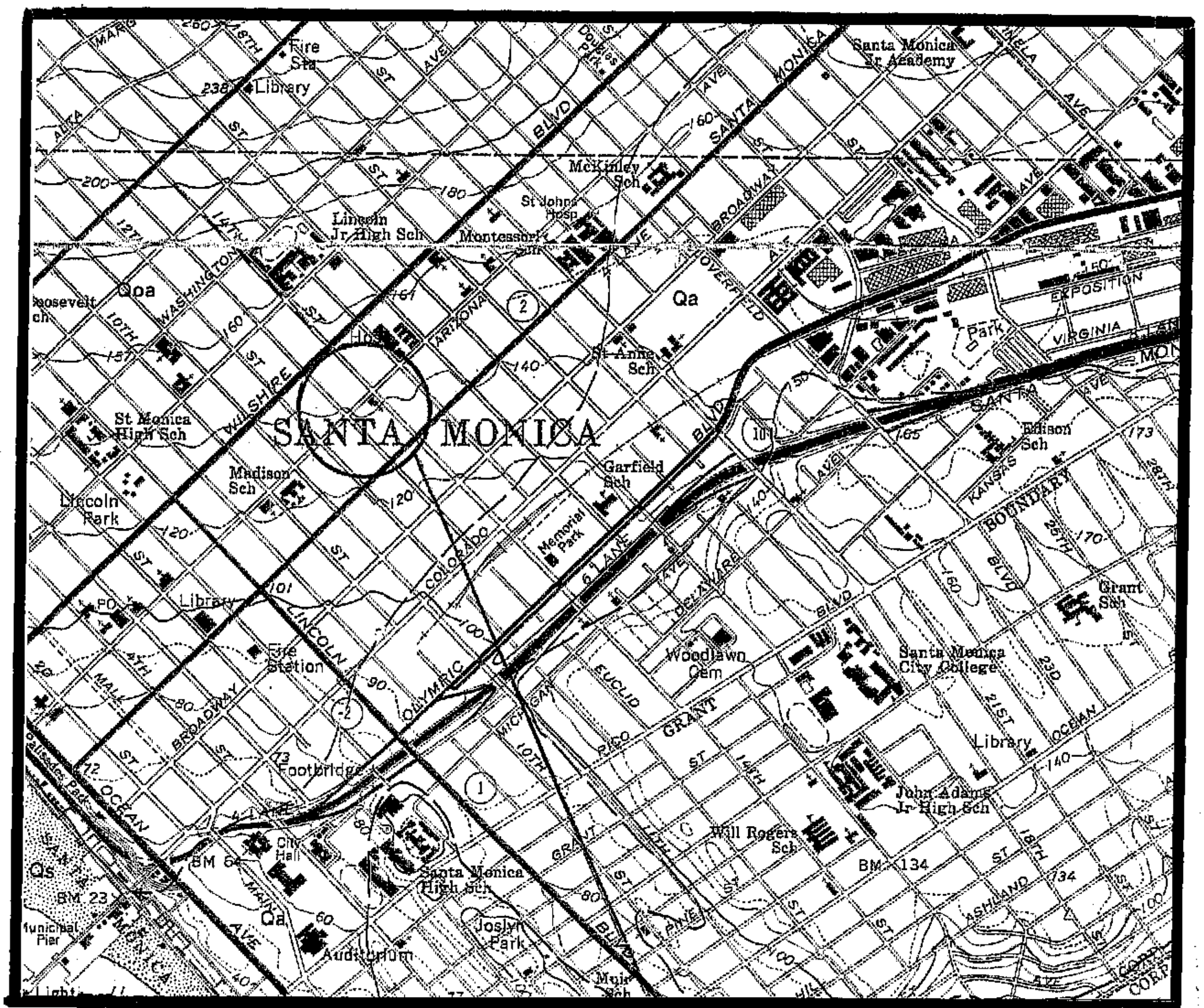
Geologic Cross Sections A-A' and B-B'

Figure 4

CYG-06-4508



Legend

Subject Site

Qoa	OLDER SURFICIAL SEDIMENTS *older alluvium of gray to light brown pebble-gravel, sand and silt-clay* derived from Santa Monica Mountains; slightly consolidated; in *Baldwin Hills designated as Baldwin Hills sandy gravel by Weber* et al., 1982, where it is much dissected and eroded

N

Scale 1" = 2000'

(Adopted from Geologic Map of the Beverly Hill and Van Nuys (South 1/2) Quadrangles, Los Angeles County, California, Dibblee, 1991)

C. Y. GEOTECH, INC. Engineering Geology and Geotechnical Engineering	**Regional Geologic Map**	
	CYG-06-4508	**Figure 5**

5.2 Engineering Property

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

Field Dry Density (Qoa, ≤ 2ft):	92 - 107 pcf
Field Dry Density (Qoa, ≥ 2 ft):	101 - 124 pcf
Field Moisture Content (Qoa):	6 - 19 %
Maximum Dry Density (Qoa):	128 pcf
Optimum Moisture Content (Qoa):	8 %
Cohesion (Qoa, peak):	290 - 500 psf
Cohesion (Qoa, ultimate):	260 - 420 psf
Friction Angle (Qoa, peak):	28 - 34 deg
Friction Angle (Qoa, ultimate):	25 - 33.5 deg
Compressibility (Qoa):	See Plates CS-1 through CS-9
Hydroconsolidation (Qoa, ≤ 1 ft):	5.3 %
Hydroconsolidation (Qoa, > 1 ft):	0 - 0.9 %
Expansion Index (Qoa):	EI = 15

5.3 Chemical Test

The bulk sample of onsite soil was delivered to Geosystems, Inc. for chemical tests. The purposes of the chemical tests are to determine the corrosivity of onsite soil. The results of the chemical tests are presented in Appendix A and summarized in the following table.

Soil Type	PH Value	Chloride ppm	Sulfate ppm	Resistivity ohm-cm	Potential of Corrosion Effect
brown silty sand	8.3	150	16.2	25912	mildly corrosive

Certain sulfate minerals present in the soil, rockmass and groundwater have a detrimental effect on concrete. When soluble sulfate concentrations are greater than 1,000 ppm in soil and 150 ppm is groundwater, mitigation measures must be taken to protect any concrete structures in contact with the soils. The soil tested has a sulfate concentration of 16.2 ppm which is much less than the critical concentration for sulfate in soil.

Large concentrations of chlorides will adversely affect ferrous materials, such as, iron and steel. When chloride concentrations exceed 18,000 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 soil tested has a chloride concentration of 150 ppm which is much less than the critical value of chloride concentration.

Mitigation measures must be taken to protect concrete and steel in soil when the PH value gets down around 4. The PH test indicated a PH value of 8.3 for the tested soil. A soil with a PH value of 8.3 will not have adverse effect of soil corrosion.

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. The test indicated an electrical resistivity of 25912 ohm-cm for the tested soil. A soil with an electrical resistivity of 25912 ohm-cm is considered as a mild corrosive soil and will not have adverse effect of soil corrosion.

Soil Resistivity, ohm-cm	0 - 1,000	1,000 - 2,000	2,000 - 10,000	over 10,000
Corrosivity Category	severely corrosive	corrosive	moderately corrosive	mildly corrosive

Based on the results of the chemical test, it is our opinion that onsite soil is mildly corrosive and will not have adverse effect of soil corrosion.

6.0 SURFACE AND SUBSURFACE WATER

No surface water or ponding water was observed on the site during field exploration. Surface water is limited to the precipitation falling directly onto the site.

No seeps, springs or groundwater were encountered in the borings. As shown on the CDMG (California Division of Mining and Geology) Historically Highest Ground Water Contour Map of Beverly Hills Quadrangle (see Figure 6), the historically highest groundwater underlying the site is deeper than 40 feet below the existing ground surface. In our opinion, the groundwater underlying the site is at depth and does not appear to be close enough to the surface to significantly affect the stability of the site.

7.0 FAULTING AND SEISMICITY STUDY

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

7.1 Faulting Study

The faulting study indicated that the site is not located within any of the mapped Alquist-Priolo Special Studies Zones and no fault trace of any known active or potentially active fault passes through the site. However, the site, as all of the Southern California areas, is located within a seismically active region and will experience slight to very intense ground shaking as the result of movement along various active faults in the region.

Twenty nine (29) 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 Santa Monica-Hollywood fault, Arroyo Parida-More Ranch fault, Newport-Inglewood fault, Malibu Coast fault, Palos Verdes fault and Elysian Park Seismic Zone fault.

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

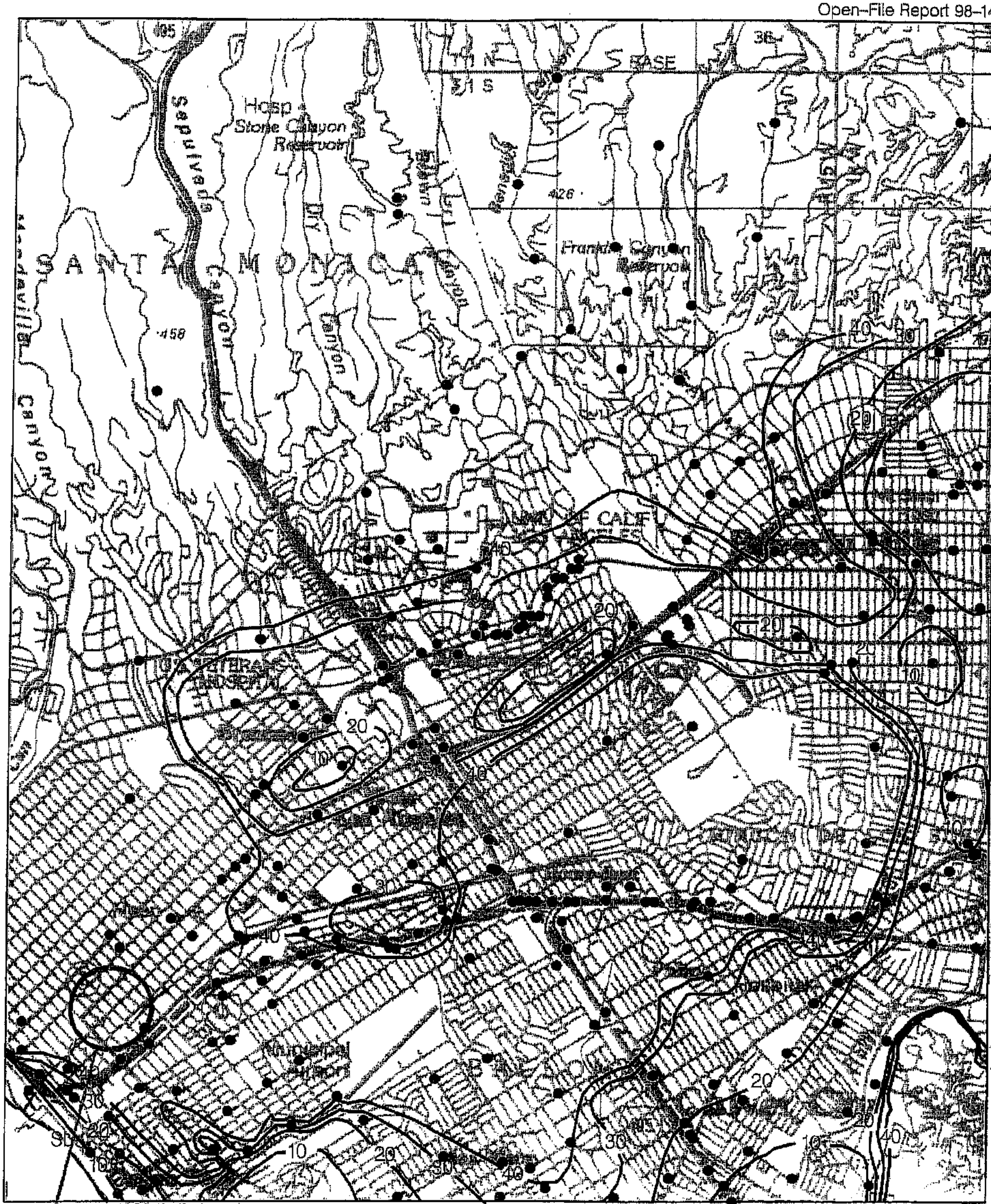


Plate 1.2 Historically Highest Ground Water Contours and Borehole Log Data Locations, Beverly Hills Quadrangle.

● Borehole Site

30 Depth to ground water in feet

X Site of historical earthquake-generated liquefaction. See "Areas of Past Liquefaction" discussion in text.

Subject Site

ONE MILE
SCALE

Figure 6

7.2 Seismicity Study

The seismicity study indicates that the largest credible and probable peak ground accelerations (mean (m) + 1 standard deviation (σ)) which may impact the site are 0.96g (g:gravity) and 0.61g, respectively. The largest credible and probable repeatable high ground accelerations (m+σ) which may impact the site are 0.62g and 0.40g, respectively. The peak and repeatable high ground accelerations (m+σ) for a 50-year exposure and 10% exceedance are approximately 0.52g and 0.34g, respectively. The maximum credible magnitude, peak ground acceleration and repeatable high ground acceleration which may impact the subject site caused by the most significant fault systems and the San Andreas 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
Santa Monica-Hollywood fault	2	7.5	0.96g	0.63g
Arroyo Parida-More Ranch fault	2	7.5	0.10g	0.07g
Newport-Inglewood fault	7	7	0.62g	0.40g
Malibu Coast fault	7	7.5	0.82g	0.53g
Palos Verd-Coron,B.-A.Blan fault	12	7.5	0.56g	0.36g
Elysian Park Seismic Zone	17	7	0.43g	0.28g
San Andreas fault	68	8.3	0.17g	0.17g

7.3 Seismic Factors

As shown on Figure 7, the subject site is located within the 2-kilometer near-source zones of Santa Monica-Hollywood fault. The seismic factors listed in the following table for structural design were determined based on the findings of data research and seismic evaluation and in accordance with California Building Code, Uniform Building Code and CDMG Active Fault Near-Source Zone Maps.

Seismic Factor	Value	Reference
Seismic Zone	Zone 4	Figure 16-2, 1997 UBC
Seismic Zone Factor	0.40	Table 16-I, 1997 UBC
Soil Profile Type	Sd	Table 16-J, 1997 UBC
Seismic Source Type (Santa Monica - Hollywood Fault)	B	Table 16-U, 1997 UBC
Near Source Factor, Na (Santa Monica - Hollywood Fault)	1.3	Table 16-S, 1997 UBC
Near Source Factor, Nv (Santa Monica - Hollywood Fault)	1.6	Table 16-T, 1997 UBC
Seismic Response Coefficient, Ca	0.44 Na	Table 16-Q, 1997 UBC
Seismic Response Coefficient, Cv	0.64 Nv	Table 16-R, 1997 UBC

7.4 Santa Monica Fault and Future Seismic Activity

The Santa Monica fault and associated fault traces are buried under unconsolidated alluvial sediments and is not exposed anywhere at the ground surface. For this reason, the precise location of the fault is not known. While researchers consider the Santa Monica fault to be potentially active, it has yet to be zoned active by

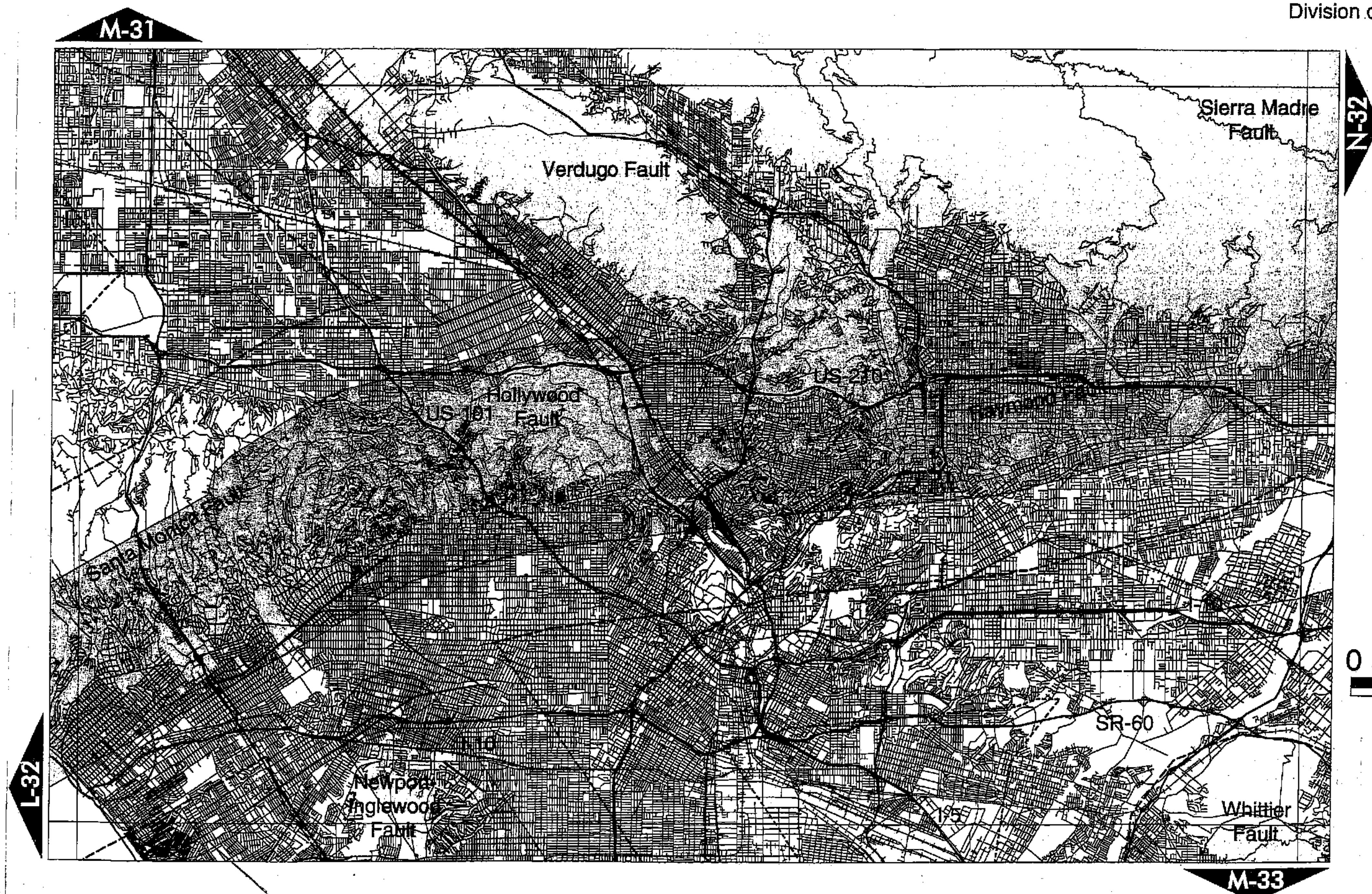
M-32

Active Fault Near-Source Zones

This map is intended to be used in conjunction with the 1997 Uniform Building Code, Tables 16-S and 16-T

M-32

California Department of Conservation
Division of Mines and Geology



Subject Site

LEGEND

See expanded legend and index map

Shaded zones are within 2 km of known seismic sources.

A fault

B fault

Contours of closest horizontal distance to known seismic sources.

5 km

10 km

15 km

0 5 10

Kilometers

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Active Fault Near-Source Zone

Figure 7

CYG-06-4508

the State of California. The Santa Monica Faults is an oblique/left-lateral fault which exhibits pronounced near-surface strain which has caused development of a series of near-vertical, left-lateral strike slip faults and a near-surface blind thrust. Work performed in 1992 and 1993 by James F. Dolan and Kerry Sieh for the Caltech Seismological Laboratory indicate that at least three surface-rupturing earthquakes have occurred on one of the fault strands during the past 12,000 to 15,000 years. The predicted recurrence interval is approximately 4,000 to 5,000 years. While the researchers consider the Santa Monica Fault to be active, the fault has not yet been zoned active by the State of California.

The homeowners however should be aware that the Santa Monica fault is considered active and that there exists a risk for future seismic activity along the fault during the occupancy in the property. The risk is considered low however based upon the long recurrence interval (approximately 7000 to 8000 years) as presented in the recent professional publication by Dolan, et. Al., 2000. Should a seismic event occur on the Santa Monica Fault, there exists a higher than average risk of fault repture at the site. Construction solutions to reduce the risk of damage in the event of fault repture are nor considered to be effective. The future homeowners are advised to obtain earthquake insurance to help mitigate the cost of damage repairs to the structure during future seismic events.

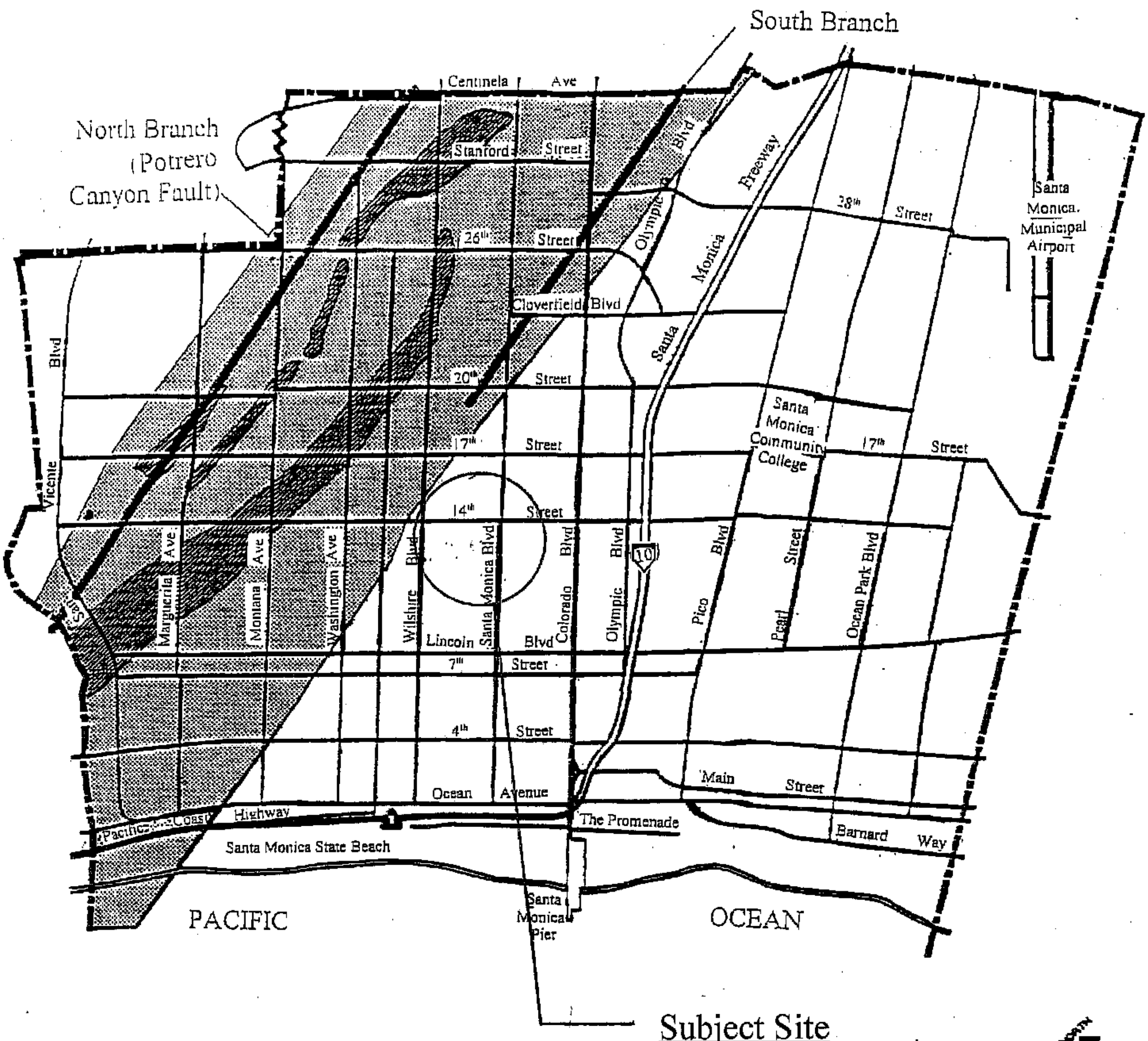
<u>7.5 City of Santa Monica Safety Element</u>
As shown on Figure 8 adopted from the City of Santa Monica Safety Element Map, the site is not located within the City of Santa Monica Fault Hazard Management Zone, Published Fault Location Zones or Geomorphic Fault Scarp Zones. The Fault Hazard Management Zone was defined by the Seismic Concerns Section of the Safety Element Environmental Impact Report for the City of Santa Monica by Cotton/Beland Associates, Inc.

As shown on Figure 9 adopted from the City of Santa Monica Safety Element Map, the site is not located any potential liquefaction zone as mapped in the City of Santa Monica liquefaction zone map. As shown on Figure 10, the site is not located within any of the liquefaction potential zones as mapped in the CDMG (California Division of Mines and Geology) Seismic Hazard Maps.

<u>7.6 Earthquake-Induced Geologic Hazards</u>
Based on the findings of field exploration, faulting and 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 below:

<u>7.6.1 Liquefaction</u>
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 four major effects: 1) the consolidation of loose sediments with resultant settlement of the ground surface, 2) lateral sliding or spreading, and 3) sand boiling.

Liquefaction susceptibility under a given earthquake is related to the gradation and relative density characteristics of the soil, the in-situ stresses prior to ground motion, and the depth to the water table, as well



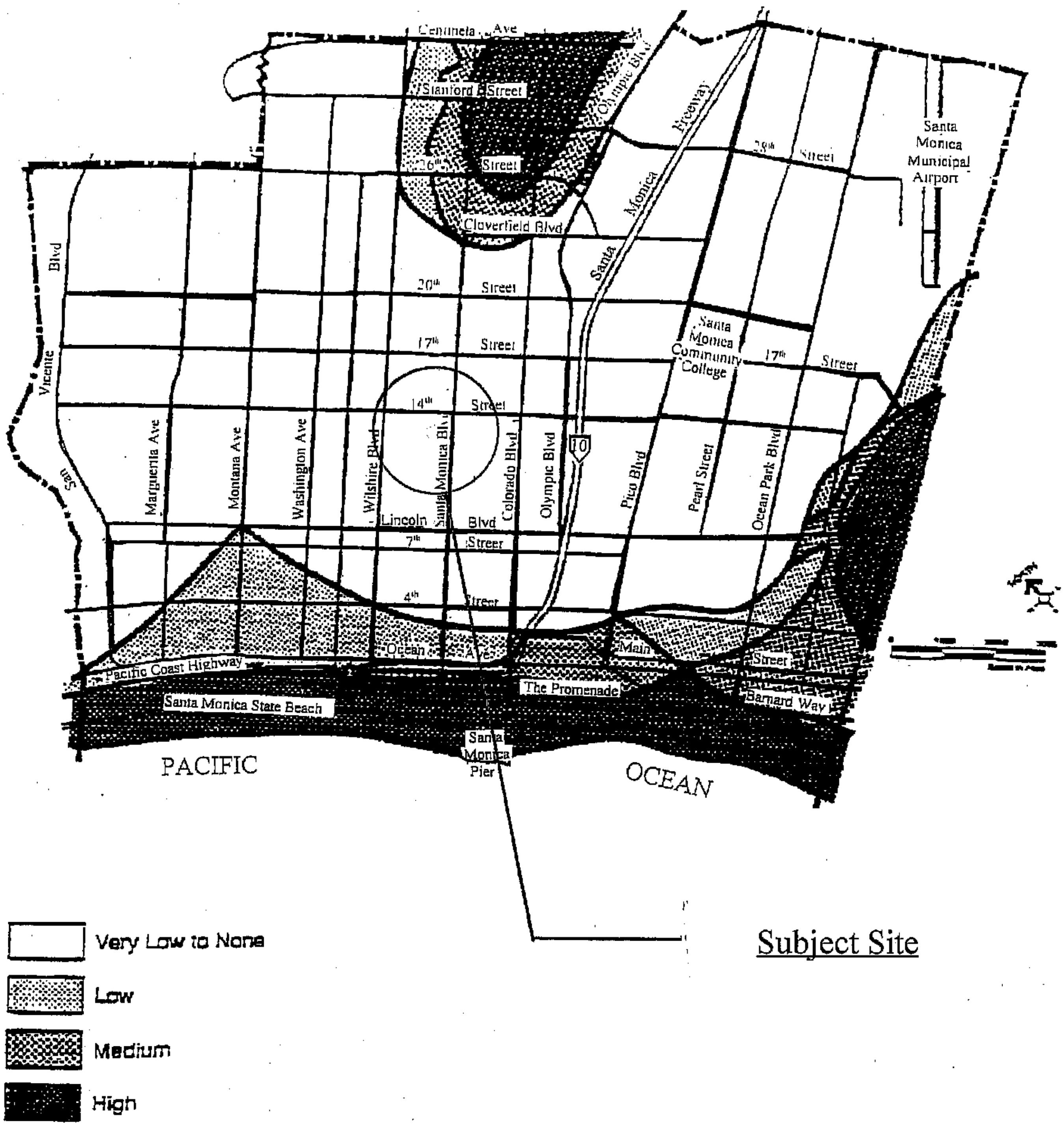
Published Fault Locations

Geomorphic Fault Scarps

Fault Hazard Management Zone

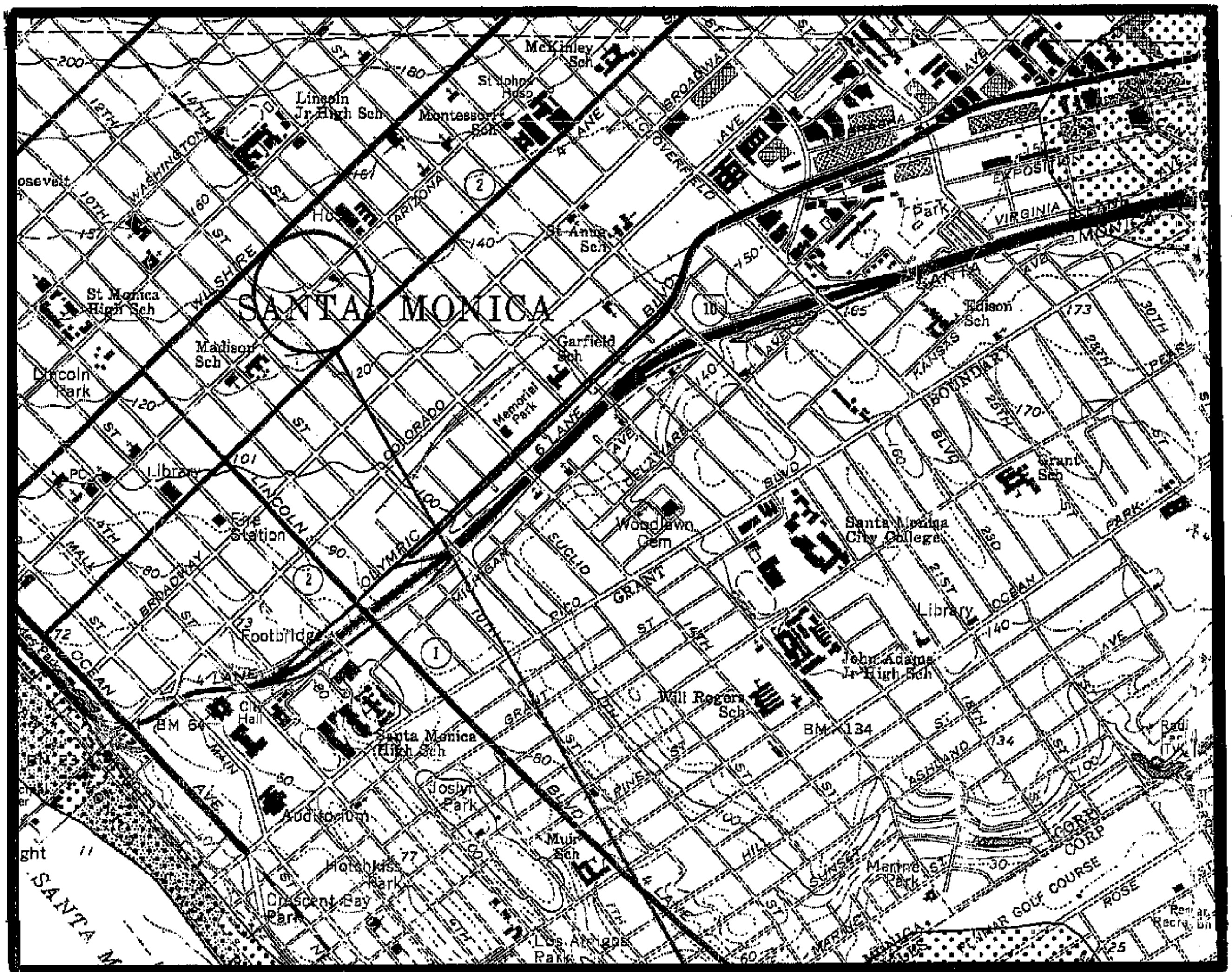
(Adopted from Santa Monica Safety Element, 1995)

C. Y. GEOTECH, INC. Engineering Geology and Geotechnical Engineering	City of Santa Monica Hazard Management Zone	
	CYG-06-4508	Figure 8



(Adopted from Santa Monica Safety Element. 1995)

C. Y. GEOTECH, INC. Engineering Geology and Geotechnical Engineering	City of Santa Monica Liquefaction Potential Zones	
	CYG-06-4508	Figure 9



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Subject Site

Liquefaction

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

Earthquake-Induced Landslide

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

N

Scale 1" – 2000'

(Adopted from State of California Seismic Hazard Zones.)

C. Y. GEOTECH, INC. Engineering Geology and Geotechnical Engineering	**Seismic Hazard Map**
CYG-06-4508	**Figure 10**

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 not located within any of the liquefaction potential zone as mapped in the City of Santa Monica Liquefaction Potential Zone Map. As shown on Figure 10, the site is not located within any of the liquefaction potential zones as mapped in the CDMG Seismic Hazard Maps. In our opinion, the potential of liquefaction within the site is low due to the depth to groundwater and the competency of older alluvium.

7.6.2 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 7.1, 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 passes through the site. As shown on Figure 8, the site is not located within any of the published fault locations and mapped Geomorphic Fault Scarp Zones. No fault trace passes through the site. In our opinion, the potential of on-site ground rupture or cracking due to shaking from local seismic events is low.

7.6.3 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 subject site is essentially flat and is not subjected to liquefaction. Therefore, the site is not subjected to earthquake-induced landsliding or lateral spreading.

7.6.4 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. Suitable site processing can eliminate compressible materials of low relative density and, thereby, will tend to reduce the potential for ground lurching.

7.6.5 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.

7.6.6 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 strength of soils and the occurrence of groundwater. Laboratory tests indicated that the relative compaction of onsite older alluvium below 8 feet is in the range of 85% to 97% and the compressibility of the older alluvium is low. In our opinion, onsite shallow soils may be subjected to minor earthquake-induced settlement. However, the settlement will not affect the integrity and competency of the building structure due to the fact that the proposed structure will be supported by competent alluvium a minimum of 8 feet below the existing ground surface.

8.0 SLOPE STABILITY

The site is essentially flat. No evidence of deep-seated slope failure or other type slope failure was observed within the site during our field exploration. The site is not located within any of the landslide areas mapped in the available public geologic maps. As shown on Figure 10, the site is not located within any of the earthquake-induced landslide zones mapped in the CDMG Seismic Hazard Maps. In our opinion, the site is free from the potential of landslide.

A wedge slope stability analysis using the Free Body Diagram method was performed to calculate the equivalent fluid pressure required for the design of a 11-foot high basement retaining wall. The lowest ultimate shear strength parameters of the older alluvium within excavation depths were used in the analysis. The analysis indicated that a triangular-distributed equivalent fluid pressure of 30 pounds per square foot per foot of depth (psf/ft) or a trapezoidal-distributed lateral force of 19H pounds per square foot per linear foot of width can be used in the design of the basement retaining wall (see Figures 11 and 12). H is the retaining height of the basement wall. The results of the analysis are presented in Appendix B.

Three wedge slope stability analyses using the Free Body Diagram method were performed to calculate the stability of the following temporary excavation: 1) a 11-foot high vertical high temporary excavation, 2) a 13-foot high vertical temporary excavation, and 3) 13-foot high temporary excavation with a surcharge of 2000 pounds per linear foot of width. The lowest peak shear strength parameters of the older alluvium within excavation depths were used in the analyses. The analyses indicated the following findings: 1) a factor of safety greater than the minimum code requirement for 11-foot high temporary excavation, 2) a factor of safety less than the minimum code requirement for 13-foot high temporary excavation, 3) an equivalent fluid pressure of 22 psf/ft to achieve a factor of safety of 1.25 for 13-foot high temporary excavation with a surcharge of 2000 pounds per linear foot of width. The results of the analyses are presented in Figure 13 through Figure 15.

Two slot cut calculations were performed for a 13-foot high and 8-foot wide A/B/C slot cut with no surcharge and a 13-foot high and 8-foot wide A/B/C slot cut with a surcharge of 2000 pounds per linear foot. The lowest peak shear strength parameters of the older alluvium within excavation depths were used in analyses. The calculation indicated factors of safety greater than the minimum code requirement for both cases. The results of the analyses are shown in Figures 16 and 17.

9.0 CONCLUSIONS AND RECOMMENDATIONS

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

Conventional spread footings founded into competent older alluvium which is approximately 2 feet below the existing ground surface can be used to support the proposed condominium. Alternatively, deep foundation such as end bearing caissons or skin friction piles entirely founded into competent older alluvium can also be used to support the proposed condominium.

Equivalent Fluid Pressure (Free Body Diagram Method)

Program Made by C. Y. Geotech, Inc.

Project Name:

CYG-06-4508 11 feet Retaining Wall / Level Backfill (Alluvium)

GEOMETRY OF CRITICAL ACTIVE WEDGE:

Height of the Retaining Wall	=	11 feet
Slope Angle of Retained Slope	=	0 degree
Dip Angle of Critical Wedge	=	54 degree

SHEAR STRENGTH PARAMETERS:

Unit Weight	=	131 pcf
Cohesion	=	350 psf
Friction Angle	=	25 degree
Mobilized Cohesion	=	233.8 psf
Mobilized Friction Angle	=	17.3 degree

REQUIRED FACTOR OF SAFETY = 1.5

RESULTS

Dip Angle of Critical Slip Surface	=	54 degree
Total Weight of Active Wedge	=	5758 lbs
Frictional Resistance (Cm * L)	=	3173 lbs
Required External Force for Wall	=	517 lbs
Required Equivalent Fluid Pressure	=	8.5 psf/ft

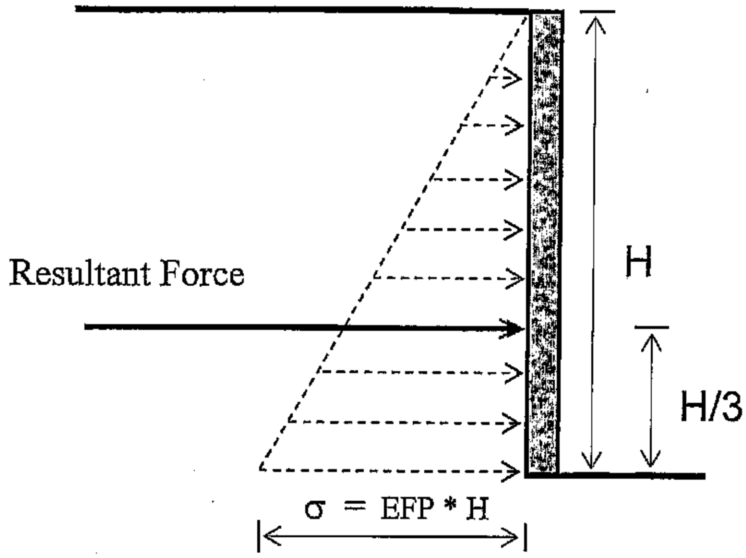
** Rankine Wedge is not the most critical wedge **

RECOMMENDED EFP VALUE:

Triangular-Distributed EFP	=	30 psf/ft
Trapezoidal-Distributed EFP	=	EFP(Tri)/1.6
	=	19 H psf/ft

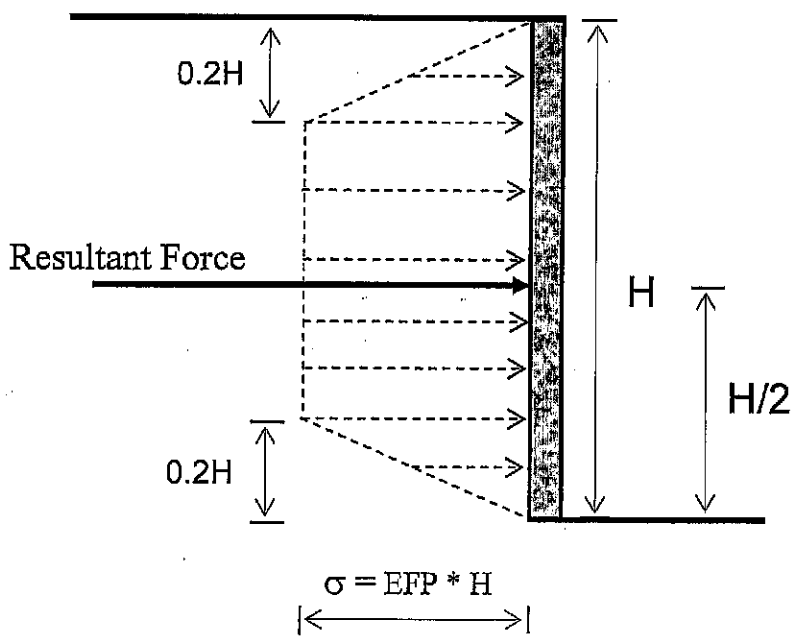
Figure 11

Nonrestrained Wall



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

Restrained Wall



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

C. Y. GEOTECH, INC. Engineering Geology and Geotechnical Engineering	**Triangular and Trapezoidal Distribution Diagram of Lateral Force**	
	CYG-06-4508	**Figure 12**

Equivalent Fluid Pressure (Free Body Diagram Method)

Program Made by C. Y. Geotech, Inc.

Project Name:

CYG-06-4508 11 feet Temporary Cut / Level Backfill (Alluvium)

GEOMETRY OF CRITICAL ACTIVE WEDGE:

Height of the Retaining Wall	=	11 feet
Slope Angle of Retained Slope	=	0 degree
Dip Angle of Critical Wedge	=	58 degree

SHEAR STRENGTH PARAMETERS:

Unit Weight	=	125 pcf
Cohesion	=	290 psf
Friction Angle	=	31 degree
Mobilized Cohesion	=	232.5 psf
Mobilized Friction Angle	=	25.7 degree

REQUIRED FACTOR OF SAFETY = 1.25

RESULTS

Dip Angle of Critical Slip Surface	=	58 degree
Total Weight of Active Wedge	=	4726 lbs
Frictional Resistance (Cm * L)	=	3009 lbs
Required External Force for Wall	=	-219 lbs
Required Equivalent Fluid Pressure	=	-3.6 psf/ft

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

Figure 13

Equivalent Fluid Pressure (Free Body Diagram Method)

Program Made by C. Y. Geotech, Inc.

Project Name:

CYG-06-4508 13 feet Temporary Cut / Level Backfill (Alluvium)

GEOMETRY OF CRITICAL ACTIVE WEDGE:

Height of the Retaining Wall	=	13 feet
Slope Angle of Retained Slope	=	0 degree
Dip Angle of Critical Wedge	=	58 degree

SHEAR STRENGTH PARAMETERS:

Unit Weight	=	125 pcf
Cohesion	=	290 psf
Friction Angle	=	31 degree
Mobilized Cohesion	=	232.5 psf
Mobilized Friction Angle	=	25.7 degree

REQUIRED FACTOR OF SAFETY = 1.25

RESULTS

Dip Angle of Critical Slip Surface	=	58 degree
Total Weight of Active Wedge	=	6600 lbs
Frictional Resistance (Cm * L)	=	3556 lbs
Required External Force for Wall	=	384 lbs
Required Equivalent Fluid Pressure	=	4.5 psf/ft

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

Figure 14

Equivalent Fluid Pressure (Free Body Diagram Method)

Program Made by C. Y. Geotech, Inc.

Project Name: CYG-06-4508 13 feet Temporary Cut with Surcharge

GEOMETRY OF CRITICAL ACTIVE WEDGE:

Height of the Retaining Wall	=	13 feet
Height of the Slope Above Wall	=	0 feet
Slope Angle of Retained Slope	=	0 degree
Dip Angle of Critical Wedge	=	64 degree

SHEAR STRENGTH PARAMETERS:

Unit Weight	=	125 pcf
Cohesion	=	290 psf
Friction Angle	=	31 degree
Mobilized Cohesion	=	232.5 psf
Mobilized Friction Angle	=	25.7 degree

REQUIRED FACTOR OF SAFETY = 1.25

Vertical Load due to Surcharge:	=	2000 lbs
Additional Lateral Resistance From Front Wedge	=	0 lbs

RESULTS

Dip Angle of Critical Slip Surface	=	64 degree
Total Weight of Active Wedge	=	7152 lbs
Frictional Resistance (Cm * L)	=	3356 lbs
Required External Force for Wall	=	1798 lbs
Required Equivalent Fluid Pressure	=	21.3 psf/ft

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

Figure 15

Slot Cut Calculation

Purpose: Stability of a 13-foot High & 8-foot Wide Slot Cut
Earth Material : Alluvium

Geometry Input:

Height of Slot Cut [H]	=	13	ft
Spacing of Slot Cut [S]	=	8	ft
Surcharge [q]	=	0	lbs per linear foot of width

Soil Parameters Input:

Unit Weight [γ]	=	125	pcf
Cohesion [C]	=	290	psf
Friction Angle [ϕ]	=	31	degrees

Results:

Dip Angle of Potential Slip Surface [δ]	$= 45 + \phi/2$	=	60.5 degrees
Length of Potential Slip Surface [L]	= H / Cos (90-δ)	=	14.9 feet
Weight of Potential Slip Mass [W]	= 0.5 x H x H x Tan(90-δ) x γ x S	=	47808 lbs
Weight (with Surchage) [Ws]	= W+q*S	=	47808 lbs
Sliding Force [SF]	= Ws x Sin(δ)	=	41610 lbs
Resisting Force (1) [RF1]	= Ws x Cos(δ) x Tan (ϕ)	=	14145 lbs
Resisting Force (2) [RF2]	= C x L x S	=	34652 lbs
Resisting Force (3) [RF3]	= C x 0.5 x H x H x Tan (90-δ) x 2	=	27729 lbs

Factor of Safety (FS)

= (RF1 + RF2 + RF3) / SF = 1.84 > 1.25 O.K.

Figure 16

Slot Cut Calculation

Purpose: Stability of a 13-foot High & 8-foot Wide Slot Cut
Earth Material : Alluvium

Geometry Input:

Height of Slot Cut [H]	=	13	ft
Spacing of Slot Cut [S]	=	8	ft
Surcharge [q]	=	2000	lbs per linear foot of width

Soil Parameters Input:

Unit Weight [γ]	=	125	pcf
Cohesion [C]	=	290	psf
Friction Angle [ϕ]	=	31	degrees

Results:

Dip Angle of Potential Slip Surface [δ]	$= 45 + \phi/2$	=	60.5 degrees
Length of Potential Slip Surface [L]	$= H / \mathrm{Cos}(90-\delta)$	=	14.9 feet
Weight of Potential Slip Mass [W]	$= 0.5 \times H \times H \times \mathrm{Tan}(90-\delta) \times \gamma \times S$	=	47808 lbs
Weight (with Surchage) [Ws]	= W+q*S	=	63808 lbs
Sliding Force [SF]	$= Ws \times \mathrm{Sin}(\delta)$	=	55535 lbs
Resisting Force (1) [RF1]	$= Ws \times \mathrm{Cos}(\delta) \times \mathrm{Tan}(\phi)$	=	18879 lbs
Resisting Force (2) [RF2]	$= C \times L \times S$	=	34652 lbs
Resisting Force (3) [RF3]	$= C \times 0.5 \times H \times H \times \mathrm{Tan}(90-\delta) \times 2$	=	27729 lbs

Factor of Safety (FS)

= (RF1 + RF2 + RF3) / SF = 1.46 > 1.25 O.K.

Figure 17

The temporary excavations for the construction of the basement will remove vertical and lateral supports of adjacent properties and public streets. Therefore, A/B/C slot cut method and/or shoring system will be required for the temporary excavations.

9.1 111 Statement

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

9.2 Site Preparation

The upper 2 feet of existing soils, at their present condition, are not suitable for the support of concrete slabs. If concrete slabs are to be supported by compacted fill, the existing soils in concrete slab areas should be removed to a depth of 2 feet below the existing ground surface and then recompacted to be compacted fill for slab support. The removal and recompaction for interior concrete slabs can be limited to surrounding footings. The removal and recompaction for exterior concrete slabs should be extended horizontally a minimum of 2 feet beyond the boundaries of concrete slabs in all directions.

9.3 Conventional Spread Footings

Conventional spread footings founded into competent older alluvium which is approximately 2 feet below the existing ground surface can be used to support of the proposed condominium. The following recommendations can be used in the design of conventional spread footings.

a. Conventional spread footings should be entirely supported by competent older alluvium which is approximately 2 feet below the existing round surface.

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

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

d. An allowable vertical bearing pressure of 2000 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 allowable vertical bearing capacities can be increased by one-third (1/3) when considering short duration wind or seismic loads.

e. Lateral force can be resisted by frictional resistance and passive earth pressure. An allowable friction coefficient of 0.3 and an allowable passive earth pressure of 300 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 1/3. The calculations of friction coefficient and passive earth pressure are shown on Figure 18.

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

Calculation of Passive Bearing Capacity

Version 4.0

Unit Weight = 131 psf

Cohesion = 350 psf

Friction Angle = 25 degrees

Allowable Lateral Passive Earth Pressure = 300 lb/ft^2 per additional foot of footing width or embedment depth

Frictional coefficient = 0.3

$Kp = Tan^2(45+FA/2) = 2.46$

$Tan(45+FA/2) = 1.57$

$$P_p = 0.5 \times \gamma \times D^2 \times Tan^2(45+\phi/2) + 2 \times C \times D \times Tan(45+\phi/2)$$

$$F_p = Tan(\phi)$$

where:

P_p : Passive Earth Pressure

F_p : Friction Resistance

D: Embedment Depth

C: Cohesion

Embedment Depth = 1 feet Passive Earth Pressure = 300 psf

Pp = 0.5 x 131 x 1 x 1 x 2.46 + 2 x 350 x 1 x 1.57 = 1260 lbs/ft

Recommended Passive Lateral Resistance = 0.5 x 300 x 1 x 1 = 150 lbs/ft

FS for Passive Lateral Resistance = 1260 / 150 = 8.4 >1.5 OK

Embedment Depth = 5 feet Passive Earth Pressure = 1500 psf

Pp = 0.5 x 131 x 5 x 5 x 2.46 + 2 x 350 x 5 x 1.57 = 9529 lbs/ft

Recommended Passive Lateral Resistance = 0.5 x 300 x 5 x 5 = 3750 lbs/ft

FS for Passive Lateral Resistance = 9529 / 3750 = 2.54 >1.5 OK

Embedment Depth = 7.5 feet Passive Earth Pressure = 2250 psf

Pp = 0.5 x 131 x 7.5 x 7.5 x 2.46 + 2 x 350 x 7.5 x 1.57 = 17319 lbs/ft

Recommended Passive Lateral Resistance = 0.5 x 300 x 7.5 x 7.5 = 8437.5 lbs/ft

FS for Passive Lateral Resistance = 17319 / 8437.5 = 2.05 >1.5 OK

Embedment Depth = 10 feet Passive Earth Pressure = 3000 psf

Pp = 0.5 x 131 x 10 x 10 x 2.46 + 2 x 350 x 10 x 1.57 = 27126 lbs/ft

Recommended Passive Lateral Resistance = 0.5 x 300 x 10 x 10 = 15000 lbs/ft

FS for Passive Lateral Resistance = 27126 / 15000 = 1.81 >1.5 OK

Frictional Coefficient = Tan (25) / 0.3 = 1.55

Figure 18

g. Prior to the placement of steel in footing excavation, an inspection should be made by the representative of CYG to ensure that the footing excavation exposes competent older alluvium. The City Inspector should be notified to inspect and approve the footing excavation prior to pouring concrete

9.4 Settlement

The total and differential settlements of the proposed townhouse supported by conventional spread footings founded into competent older alluvium as recommended are anticipated to be within tolerable limits. Total settlement of the foundation is expected to be less than ½ inch. Differential settlement should be less than ½ inch in a span of 20 feet.

The laboratory consolidation tests indicated a hydroconsolidation potential of 0 to 0.9% for the tested soil deeper than 1 foot. The laboratory tests indicated that the dry density of the tested soil below 8 feet is greater than 85%. It is recommended Section 9.3 that conventional spread footings founded into competent older alluvium 2 feet below the existing ground surface can be used to support the proposed condominium. Therefore, the potential of foundation or footing distress caused by hydroconsolidation of bearing soil is low.

It should be noted that the evaluation of settlement is based on the assumption that the proposed building area will be provided with adequate surfaceand subsurface drainage devices and that the drainage systems will be properly and constantly maintained. Additional soil settlement caused by local bearing failure 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 in the section of Drainage Control should be incorporated into design and implemented during construction.

9.5 Basement Retaining Wall

The following recommendations can be used in the design of the basement retaining wall. The evaluations for the design of the basement wall are presented in Section 8.0.

a. Conventional spread footings founded into competent older alluvium which is approximately 2 feet below the existing ground surface can be used to support the basement retaining wall.

b. A triangular-distributed equivalent fluid pressure of 30 psf/ft or a trapezoidal-distributed lateral force of 19H psf per linear foot of width can be used in the design of a 11-foot high basement retaining wall (see Figure 11). H is the retaining height of the subterraneous building wall. A basement retaining wall is usually designed as a restrained retaining wall using the trapezoidal-distributed lateral force.

c. Any anticipated superimposed loading within a 2:1 plane projected upward from the wall bottom, except retained earth materials, should be considered as surcharge and provided for in the design. The Project Structural Engineer should incorporate any anticipated superimposed loading in the design of the basement retaining wall.

d. The basement retaining should be constructed with a perforated PVC pipe in a gravel envelope at and behind the bottom of the wall. A one-foot thick zone of clean, granular soil should be placed behind the wall to 2 feet of the finished grade. The upper 2 feet should be backfilled with less permeable soil. A sump pump may be required to pump out the water collected by the subdrain system.

e. The subdrain system for the retaining wall should be inspected and approved by the representative of CYG and the City Inspector prior to placing wall backfill.

f. Retaining wall backfill must be compacted to a minimum dry density of 90% of the maximum density as determined by ASTM Standard D-1557-02.

g. The basement retaining wall should be provided with a proper waterproofing system to prevent the migration of subsurface water to the interior side of the subterraneous building wall.

9.6 Slab-On-Grade

Concrete slabs-on-grade should be either entirely supported by compacted fill or entirely supported by competent older alluvium which is approximately 2 feet below the existing ground surface. If concrete slabs-on-grade are to be supported by compacted fill, the recommendations for soil removal and recompaction for the preparation of slab subgrade in Section 9.2 should be incorporated in the preparation of slab subgrade.

If interior concrete slab-on-grade supported by compacted fill is proposed, the upper 2 feet of existing soils in the interior concrete slab area should be removed and recompacted to be compacted fill for slab support. The removal and recompaction can be limited to surrounding footings.

In exterior concrete slab-on-grade is proposed, the existing soils in the exterior concrete slab area should be removed to a minimum depth of 2 feet below the existing ground surface and then recompacted to be compacted fill for the support of concrete slab. The removal and recompaction should be extended a minimum of 2 feet beyond the boundaries of concrete slab in all directions.

Concrete slab-on-grade should be designed for a minimum thickness of 4 inches, reinforced with No.4 bars at 16 inches on centers, both ways. Reinforcement should be properly supported to assure desired mid-height placement. A 10-mil plastic vapor barrier should be placed below the floor slabs in moisture sensitive areas. The vapor barrier should be sandwiched by two 2-inch sand layers to protect the vapor barrier from punctures and to aid in the concrete cure.

Decking, slabs and walkways are likely to experience cracking as the results of the curing process of the concrete. Shrinkage cracks are very difficult to prevent from occurring. Expansion joints are commonly installed within exterior decks in an effort to control the location of the inevitable cracks. Interior slabs however are typically not provided with expansion joint, making cracking more random. The recommended steel reinforcement is intended to reduce the severity of cracking and must be properly installed to ensure proper performance. Rigid or brittle floor covering, such as tile or marble may also experience cracking during the curing process of the concrete slab underneath and/or minor settlement. Providing a slip sheet between the slab and floor covering will help to reduce cracking of the floor covering.

9.7 Fill Placement and Soil Compaction

Fill placement and soil compaction will be required for retaining wall backfill and for the preparation of slab subgrades. All grading work and fill placement should be performed in conformance with the current grading ordinances of the City of Santa Monica. The following general guidelines can be used as a basis for quality control of fill placement and soil compaction.

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

b. The bottom to receive fill soil should be inspected and approved by the representative of CYG prior to the placement of any fill soil. The bottom to receive fill soil should also be inspected and approved by the City Inspector prior to the placement of fill soil.

the City Inspector prior to the placement of fill soil.

c. The bottom to receive fill soil 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.

d. Fill materials should be placed in controlled layers which when compacted, should not exceed 6 inches in thickness. Rock greater than 6 inches in the longest side should not be placed in compacted fill.

e. All compacted fill should be thoroughly moistened and mixed to near the optimum moisture content and then compacted to a minimum dry density 90% of the maximum dry density as determined by ASTM Standard D-1557-02.

f. At least one soil density test should be performed for every two (2) feet of vertical lift. Both sand cone method and nuclear gauge method will be required for field density tests.

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

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

9.8 Temporary Excavation

Temporary excavation up to 13 feet in vertical may be required for the construction of the basement garage. Wedge slope stability analyses using the Free Body Diagram method were performed to calculate the stability of the following temporary excavation: 1) a 11-foot high vertical temporary excavation, 2) a 13-foot high vertical temporary excavations, 3) 11-foot high temporary excavation with a surcharge of 2000 pounds per linear foot of width, 4) a 13-foot high and 8-foot wide A/B/C slot cut with no surcharge, and 5) a 13-foot high and 8-foot wide A/B/C slot cut with a surcharge of 2000 pounds per linear foot. The lowest peak shear strength parameters of the older alluvium within excavation depths were used in analyses. The results of the analysis are presented in Figures 12 through 17.

The slope stability analyses indicated the following findings: 1) a factor of safety greater than the minimum code requirement for 11-foot high temporary excavation, 2) a factor of safety less than the minimum code requirement for 13-foot high temporary excavation, 3) an equivalent fluid pressure of 22 psf/ft to achieve a factor of safety of 1.25 for 13-foot high temporary excavation with a surcharge of 2000 pounds per linear foot of width, 4) a factor of safety greater than the minimum code requirement for a 13-foot high and 8-foot wide A/B/C slot cut with no surcharge, and 5) a factor of safety greater than the minimum code requirement for a 13-foot high and 8-foot wide A/B/C slot cut with a surcharge of 2000 pounds per linear foot.

Temporary excavation below the 1:1 lines projected downward from the bottom of adjacent structures or footings will be considered as the removal of vertical and lateral support from the adjacent structures or footings. If temporary excavation removes vertical or lateral support of any adjacent structure or footings, the temporary excavation should be protected by a shoring system or be conducted using the A/B/C slot cutting method. The temporary excavations for the construction of the basement will remove vertical and lateral supports of adjacent properties and public streets. Therefore, A/B/C slot cut method and/or shoring system will be required for the temporary excavations. The recommendations in the following table can be used in the preliminary design of temporary excavations required for the subject project.

Site Condition	Height of Excavation	Horizontal : Vertical or Shoring System
Not Adjacent to Structure	0 - 11 ft	vertical
Not Adjacent to Structure	> 11 ft	vertical for lower 11 feet and 1:1 for above 11 feet
Not Adjacent to Structure	> 13 ft	additional evaluations are required
Adjacent to Structure	0 - 13 ft	13-foot high and 8-foot wide A/B/C slot cut
Adjacent to Structure	0 - 13 ft	13-foot high and 8-foot wide A/B/C slot cut
Adjacent to Structure	0 - 13 ft	soldier piles using EFP = 22 psf/ft, maximum spacing = 8 ft
Adjacent to Public Street	0 - 13 ft	13-foot high and 8-foot wide A/B/C slot cut
Adjacent to Public Street	0 - 13 ft	13-foot high and 8-foot wide A/B/C slot cut
Adjacent to Public Street	0 - 13 ft	soldier piles using EFP = 22 psf/ft, maximum spacing = 8 ft

9.8.1 A/B/C Slot Cut

If A/B/C slot cut is to be used for the excavation for the proposed basement, the following procedures can be used in the preliminary design of A/B/C slot cutting:

a. Excavate banks to a 1:1 gradient (45 degrees).

b. Excavate the vertical slots, using the A-B-C-A-B-C sequence, first excavating the "A" slots. Slot cut may be excavated to a maximum of 8 feet in width.

c. Construct the wall sections in the "A" slots. Provide proper waterproofing and backfill between the wall sections and the bank with gravel or approved compacted fill.

d. Excavate the "B" slots after the wall sections in the "A" slots have been constructed and backfilled.

e. Construct the wall sections in the "B" slots. Provide proper waterproofing and backfill between the wall sections and the bank with gravel or approved compacted fill.

f. Excavate the "C" slots after the wall sections in the "B" slots have been constructed and backfilled.

g. Construct the wall sections in the "C" slots. Provide proper waterproofing and backfill between the wall sections and the bank with gravel or approved compacted fill.

9.8.2 Soldier Piles

The following recommendations can be used in the design of soldier piles for shoring purpose.

a. An equivalent fluid pressure of 22 psf/ft should be used in the design of a 13 foot high soldier piles.

b. Soldier piles should be embedded a minimum of 8 feet into competent alluvium but not less than the depth required for adequate lateral support.

c. Solider piles can be assumed fixed at two feet below the lowest adjacent grade.

d. Allowable passive earth pressure may be computed as an equivalent fluid having a density of 300 pcf, to a maximum passive earth pressure of 3000 psf. The earth material above the fixity point should be assumed providing no lateral support. The calculations of friction coefficient and passive earth pressure are shown on Figure 18.

e. The allowable passive earth pressure may be increased by 100% for isolated piles. Piles with spacing greater than 3 times of pile diameter can be considered as isolated piles.

9.8.3 Inspection of Temporary Excavation

The design of A/B/C slot cutting and soldier pile system should be reviewed and approved by CYG. The temporary excavation for A/B/C slot cutting should be inspected by the representative of CYG. The representative of CYG should be notified to observe the temporary excavations.

Drilling of pile holes should be observed and approved by the representative of CYG prior to placing steel. The City Inspector should be notified to inspect the pile excavation prior to pouring concrete.

All excavations should be stabilized within 30 days of initial excavation. Water should not be allowed to pond on top of the excavations nor to flow toward it. It should be noted that it is your responsibility to notify CYG to inspect the temporary excavation.

9.8.4 Monitoring of Temporary Excavation

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

It is recommended that a raker system be installed to provide additional lateral support if the lateral movement of the top of the temporary excavation and/or soldier pile exceeds 1% of the height of the temporary excavation. The base of the raker system should be embedded a minimum of 2 feet below the lowest adjacent garde. A bearing capacity of 2000 psf can be used in the design of the raker base. The additional stabilization force to be provided by the raker system will be determined by CYG when a raker system is required.

It is anticipated that major yielding of the adjacent soils may occur during construction. Care should be taken that any movements associated with the yielding are not excessive to harm any underground utilities or adjacent structures. The program of monitoring lateral movement of temporary excavation and soldier pile should be agreed upon the contractors, the site surveyor, the Project Civil Engineer, the project Shoring Engineer and CYG prior to the temporary excavation.

It is recommended that the monitoring data be evaluated by CYG to ensure the stability of the temporary excavations. It is also recommended that CYG be allowed to regularly inspect the temporary excavation as work progress in order to monitor earth strain and verify that conditions assumed for design remain unchanged.

Earth materials exposed on the face of temporary excavation should be kept moist but not saturated to retard reveling and sloughing during construction. Due to low cohesion of onsite soil, lagging or gunite will be required for the soil between soldier piles. If wood lagging is used, care should be taken to fill all void spaces between the excavation face and the lagging. All timber lagging must be removed prior to permanent construction unless the timbers are properly treated.

Placing gunite on exposed cut faces is preferable and often will lead to easier construction methods. Any materials used for backfill behind the excavation walls should be free-draining. All lagging or gunite should be placed as soon as possible after the excavation is made. Due to the arching effect of the soils, a lagging pressure of 400 psf may be used for design, providing piles are not spaced larger than 8 feet on centers.

9.9 Drainage

Final grading should provide a positive drainage to divert surface water away from the building foundation and footings in non-erosive devices to the street or other acceptable areas. The building structures should be properly provided with roof gutters and down spouts. The outlets of down spouts should be either connected to area drains or be extended a minimum of 5 feet away from the building foundation and footings. Yard areas and planter areas should be provided with adequate area drains to intercept excessive surface water.

Underground utility pipes should be absolutely leak free. Landscape watering should be kept to the minimum amount required for vegetation growth. The basement retaining should be constructed with a perforated PVC pipe in a gravel envelope at and behind the bottom of the wall. A sump pump may be required to pump out the water collected by the subdrain system. The basement retaining wall should be also provided with a proper waterproofing system to prevent the migration of subsurface water to the interior side of the subterraneous building wall.

Foundation settlement caused by local bearing failure 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 and subsurface water to foundation soil, the recommendations in this section should be properly incorporated into the design and implemented during construction. The drainage systems should be designed by the Project Civil Engineer. The drainage devices should be constantly maintained to ensure proper function.

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.

The inspection and approval of surface and subsurface drainage for the foundation pad, except the retaining wall subdrain, are usually beyond the scope of service provided by geologist and soils engineer. Therefore, it is strongly recommended that the Project Civil Engineer or the one who prepares drainage control plan be notified to inspect and approve the final conditions of surface and subsurface drainage prior to the completion of the project.

10.0 OBSERVATION AND TESTING

CYG should be notified to perform the following tasks: a) review foundation plan, temporary excavation and shoring plan and drainage plan, b) inspect and approve foundation excavation, footing excavation, temporary excavation, and bottom to receive fill soil, and c) test all fill soils placed for engineering purposes.

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

11.0 LIMITS AND LIABILITY

The conclusions and recommendations submitted in this report are based on our data research, subsurface exploration, laboratory testing, engineering evaluation and engineering analysis. The nature and extent of variations in subsurface conditions may not become evident until construction. If variations appear evident, it will be necessary to reevaluate the recommendations of this report. Due to the limited area for this investigation, additional investigation including additional borings, laboratory testing and engineering evaluation may be required prior to the final design of the proposed development.

CYG has prepared this report for the exclusive use of the client and authorized agent. 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 incorporated into the design plan and the necessary steps are taken to see that the contractors carry out such recommendations in the field.

APPENDIX A
FIELD EXPLORATION AND LABORATORY TESTING

1.0 FIELD EXPLORATION

Field exploration was performed by one of our geologists on April 28, 2006 with the aid of hand laborers. Four (4) borings were drilled to a maximum depth of 21 feet at the locations as shown on Figure 2. Hand-operated augers and hand tools were used to drill borings. All borings were logged by the geologist and backfilled on the same day of field exploration. The boring logs are presented in Plates A-1 and A-2.

Ring samples of onsite earth materials were obtained by using a split-tube ring sampler. The ring samples 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 ring samples with the brass rings were further retained in plastic, close-fitting, moisture-tight containers. Bulk samples of onsite soil was collected for laboratory compaction test and expansion index test. Ring samples and bulk samples were delivered to CYG for laboratory tests.

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 earth materials were determined: 1) field density and field moisture content, 2) maximum dry density and optimum moisture content, 3) cohesion and friction angle, 4) compressibility and hydroconsolidation, and 5) expansion index.

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 D2216-98. Unit weights were determined in general accordance with ASTM Test Designation D2937-04. The results of moisture-density tests are listed in Table 2.

2.2 Direct Shear Test

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

2.3 Consolidation Tests

Nine consolidation tests were performed on ring samples to determine the compressibility and hydro-consolidation potential of older alluvium. The consolidation tests were performed in general accordance with ASTM Standard D-2435-04. The ring sample was 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 sample was tested under the following procedures: 1) the sample is placed in a loading frame under a seating pressure of 200 psf, 2) apply vertical loads to the sample in several geometric increments and record the resulting deformations at selected time intervals, 3) adds water to the test cell and records the vertical consolidation when the applied stress reaches a simulated foundation pressure (often 2000 psf) and the sample has consolidated under that pressure, 4) repeat step 2 until a loading pressure of 4000 psf or 8000 psf and record the equilibrium consolidation, 5) unload the sample to an applied stress of 1000 psf and record the rebound of the sample. The results of tests are presented in terms of percent volume change versus applied vertical stress. The results of consolidation tests are presented on Plates CS-1 through CS-9.

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 onsite alluvium. The compaction test was performed in general accordance with ASTM Test Designation D1557-02. The procedure A of compaction test was used in the subject project. The following materials and criteria were followed in test: 1) soil sample passing No.4 sieve was used in test, 2) a 4-inch mode was used in test, 3) a 10-pound hammer with a free fall distance of 18 inches was used in test, 4) five layers of soil sample were compacted in the 4-inch mode, 5) the blow for each layer of soil sample is 25. A minimum of three soil samples were performed to determine the corresponding dry density and moisture content. The results of the test are presented in terms of moisture content verses dry density to generate a compaction curve. The maximum dry density and optimum moisture content can be determined from the compaction curve. The results of the compaction 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 onsite alluvium. The expansion index test was performed in general accordance with expansion test procedures in ASTM D4829-03 to provide an assessment of the potential for expansion or heave that could be detrimental to foundation or slab performance. The following procedures were followed in the test: 1) compact the soil sample at degree of saturation between 45 and 55 percent in a 4.01-inch-diameter, 1.0-inch-high ring, 2) apply a vertical seating pressure of 144 psf to the sample, 3) add water to the test cell and saturate the soil sample, 4) record the soil expansion until the expansion of soil sample stops. The volume of swell is converted to an expansion index. Laboratory expansion index test indicated an expansion index of 15 for the tested alluvial soil. A soil with an expansion index in the range of 0 to 20 is classified as a very low expansive soil.

BORING LOG

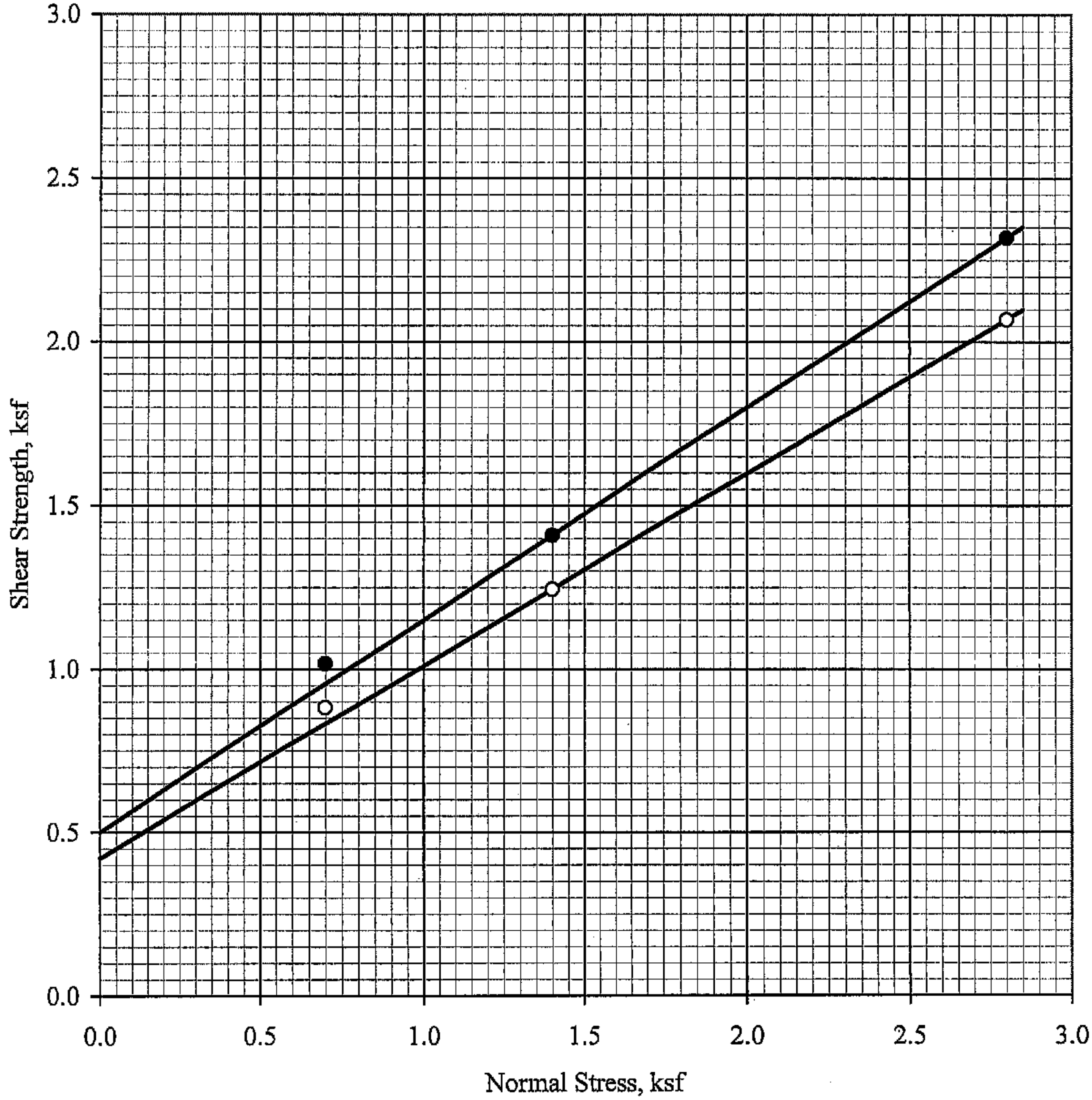
Location	Depth ft	Soils Descriptions
B - 1		**Older Alluvium (0' - 21')**
	0 - 6"	Dark brown clayey sandy silt with rootlets, slightly moist, moderately firm.
	6" - 3'	Dark brown clayey sandy silt with porous, moist, moderately firm.
	3' - 5'	Brown clayey sandy silt, slightly porous, moist, firm.
	5' - 8'	Brown clayey sandy silt to gravelly silty sand.
	8' - 11'	Brown sandy clayey silt with slightly porous, moist, firm.
	11' - 13'	Brown gravelly silty sand with slightly porous, slightly moist, dense.
	13' - 15'	Brown sandy clayey silt with slightly porous, moist, firm.
	15' - 21'	Grayish brown gravelly sand, slightly moist, dense.
		Ends at 21 ft. No caving, No water. Samples @ 2, 5, 8, 11, 14, 17, 20 ft.
B - 2		**Older Alluvium (0' - 11')**
	0 - 1'	Dark brown clayey sandy silt with rootlets, moist, moderately firm.
	1' - 3'	Dark brown clayey sandy silt with porous, moist, moderately firm.
	3' - 7'	Brown clayey sandy silt with slightly porous, occasional rock fragments, moist, firm.
	7' - 9'	Brown gravelly clayey sand with slight porous, moist, dense.
	9' - 11'	Brown gravelly clayey sand, slight porous, moist, dense.
		Ends at 11 ft. No caving, No water. Samples @ 1, 4, 7 and 10 feet.

Plate A-1

BORING LOG

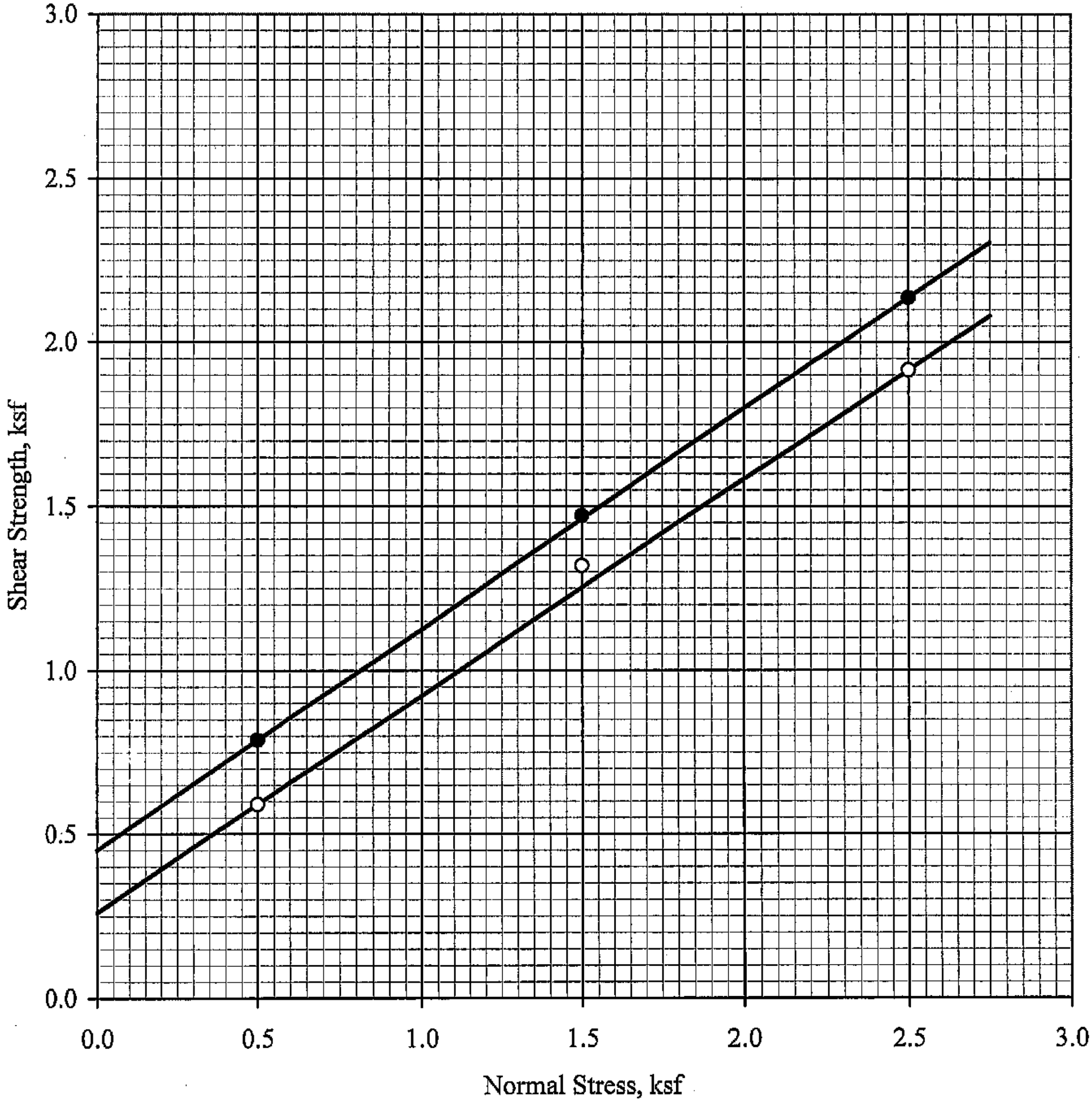
Location	Depth ft	Soils Descriptions
B - 3		**Older Alluvium (0' - 10')**
	0 - 1'	Dark brown clayey sandy silt with rootlets, moist, moderately firm.
	1' - 3'	Dark brown clayey sandy silt, porous, moist, moderately firm.
	3' - 6'	Brown clayey sandy silt with slight porous, occasional rock fragments, moist, firm.
	6' - 9'	Brown clayey sandy silt with slight porous, occasional rock fragments, moist, firm.
	9' - 10'	Brown sandy silt with slight porous, very moist to moist, moderately firm to firm.
		Ends at 10 ft. No caving, No water. Samples @ 3, 6 and 9 ft.
B - 4		**Older Alluvium (0 - 16')**
	0 - 1'	Brown clayey sandy silt with rootlets, slight moist, moderately firm.
	1' - 4'	Reddish brown clayey sandy silt with slightly porous, slightly moist, firm.
	4' - 7'	Brown sandy clayey silt with slightly porous, moist, firm.
	7' - 10'	Brown clayey sandy silt, gravels, moist, firm.
	10' - 13'	Brown gravelly silty sand, moist, moderately dense to dense.
	13' - 20'	Grayish brown gravelly sand, slight moist, dense.
		Ends at 20 ft. No caving, No water. Samples @ 1, 5, 3.5, 7.5, 10.5, 13, 15, 18 ft.

Plate A-2



● Peak	- At Saturation Moisture Content	C = 500 psf	ϕ = 33 °
○ Ultimate	- At Saturation Moisture Content	C = 420 psf	ϕ = 30.5 °

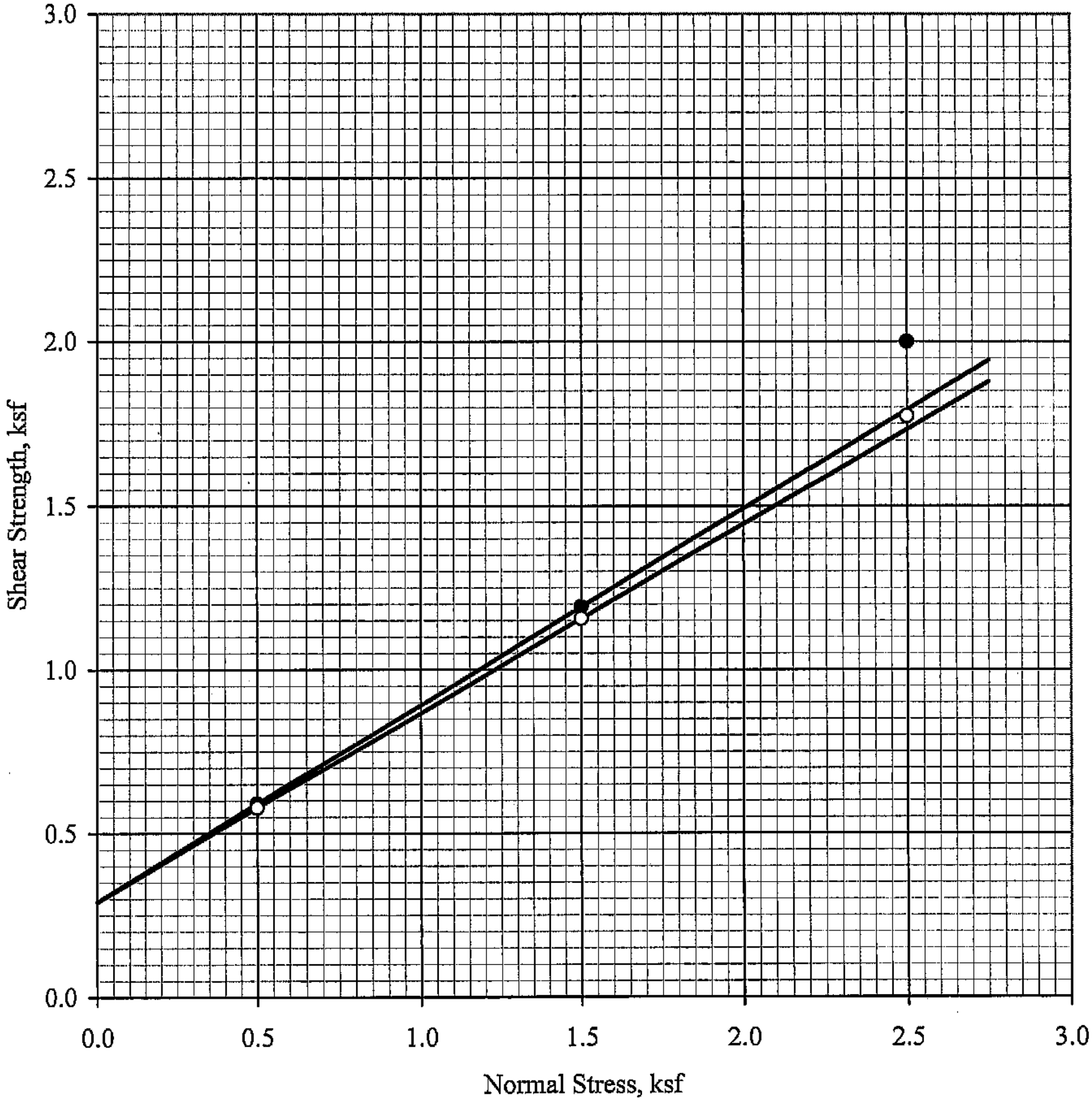
Field Dry Density = 111 pcf Field Moisture Content = 9 % Saturation Moisture Content = 19 %	Boring : B-1 Depth : 11 feet Description : Brown sandy clayey silt (Qa)
C. Y. GEOTECH, INC. Geotechnical Engineering and Engineering Geology	Vla di Tomalevski Date : 05-2006 P.N. No.: CYG-06-4508 **Shear Diagram**



● Peak - At Saturation Moisture Content C = 450 psf φ = 34 °

○ Ultimate - At Saturation Moisture Content C = 260 psf φ = 33.5 °

Field Dry Density = 120 pcf Field Moisture Content = 10 % Saturation Moisture Content = 15 %	Boring : B-1 Depth : 14 feet Description : Brown sandy clayey silt (Qa)
C. Y. GEOTECH, INC. Geotechnical Engineering and Engineering Geology	Vla di Tomalevski Date : 05-2006 P.N. No.: CYG-06-4508 **Shear Diagram**



● Peak	- At Saturation Moisture Content	C = 290 psf	ϕ = 31 °
○ Ultimate	- At Saturation Moisture Content	C = 290 psf	ϕ = 30 °

Field Dry Density = 112 pcf
Field Moisture Content = 12 %
Saturation Moisture Content = 18 %

Boring : B-2
Depth : 4 feet
Description : Brown clayey sandy silt (Qa)

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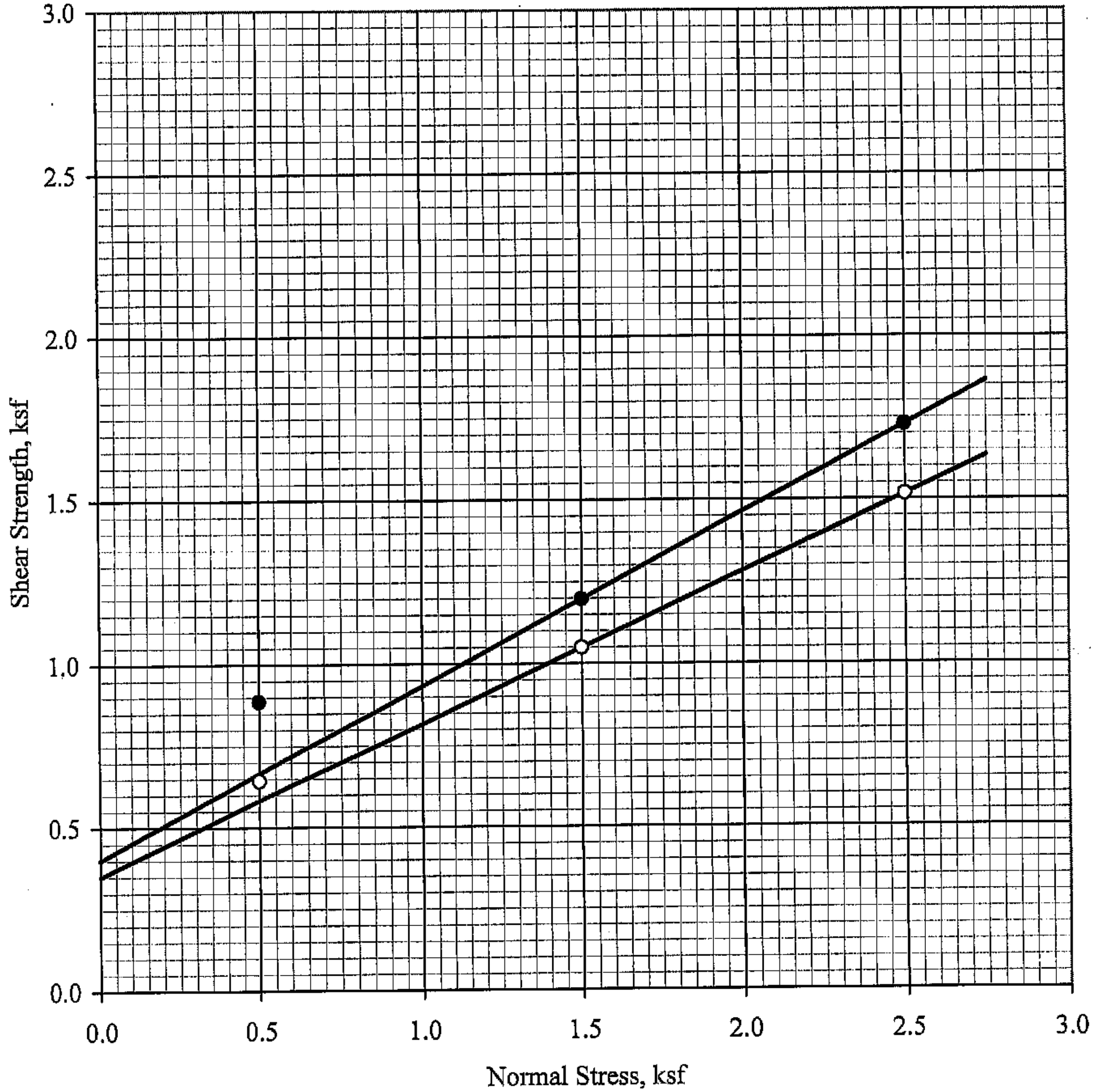
Date : 05-2006

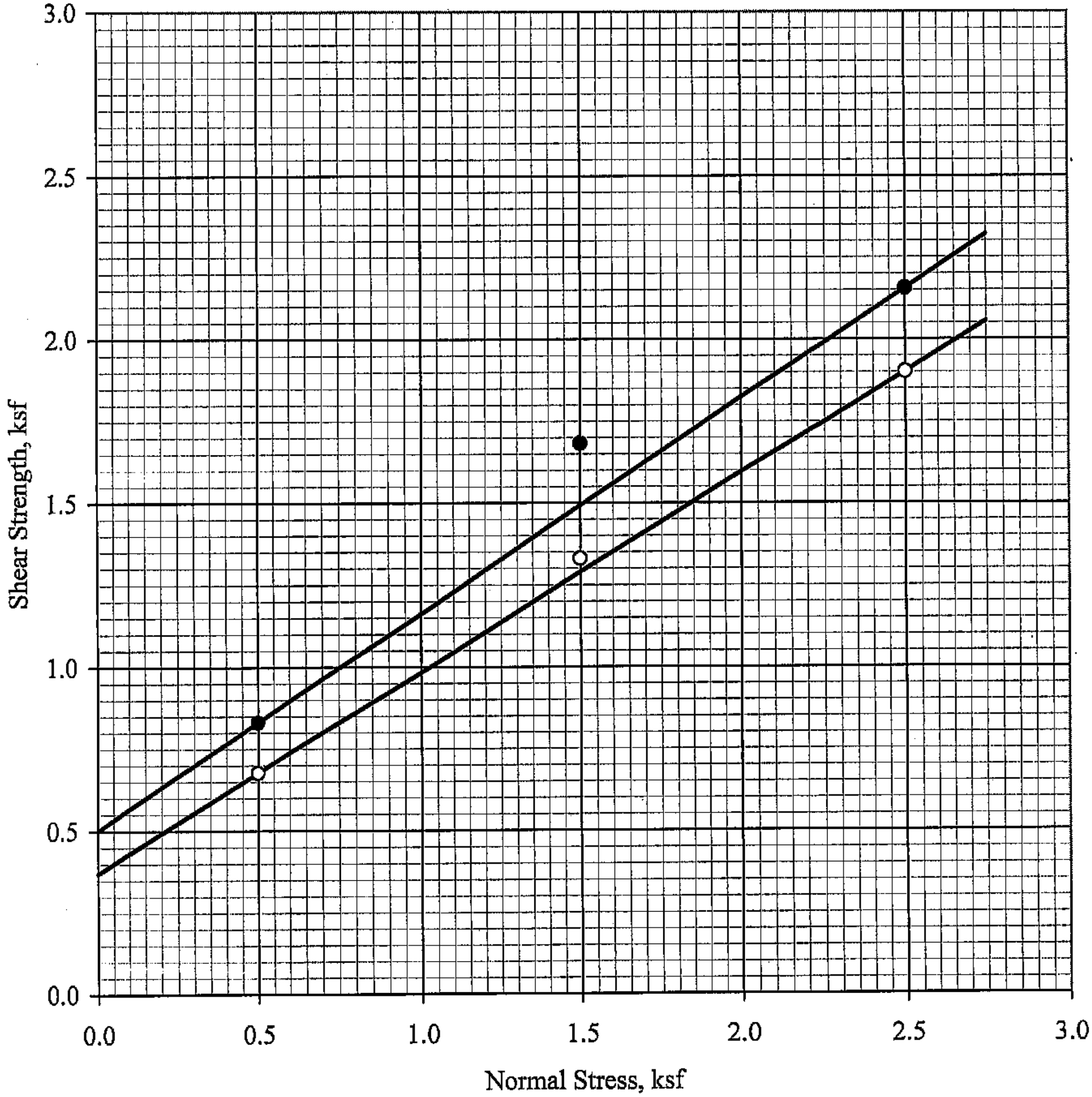
P.N. No.: CYG-06-4508

Shear Diagram



● Peak - At Saturation Moisture Content C = 400 psf φ = 28 °
○ Ultimate - At Saturation Moisture Content C = 350 psf φ = 25 °

Field Dry Density = 109 pcf Field Moisture Content = 18 % Saturation Moisture Content = 20 %	Boring : B-3 Depth : 6 feet Description : Brown clayey silt with rock fragment (Qa)
C. Y. GEOTECH, INC. Geotechnical Engineering and Engineering Geology	Vla di Tomalevski Date : 05-2006 — P.N. No.: CYG-06-4508 **Shear Diagram**



● Peak - At Saturation Moisture Content C = 500 psf ϕ = 33.5 °

○ Ultimate - At Saturation Moisture Content C = 370 psf ϕ = 31.5 °

Field Dry Density = 101 pcf Field Moisture Content = 15 % Saturation Moisture Content = 24 %	Boring : B-4 Depth : 7.5 feet Description : Brown clayey sandy silt (Qa)
C. Y. GEOTECH, INC. Geotechnical Engineering and Engineering Geology	Vla di Tomalevski Date : 05-2006 P.N. No.: CYG-06-4508 **Shear Diagram**

C. Y. GEOTECH, INC.

Geotechnical Engineering and Engineering Geology

Vladi Tomalevski

Date : 05-2006

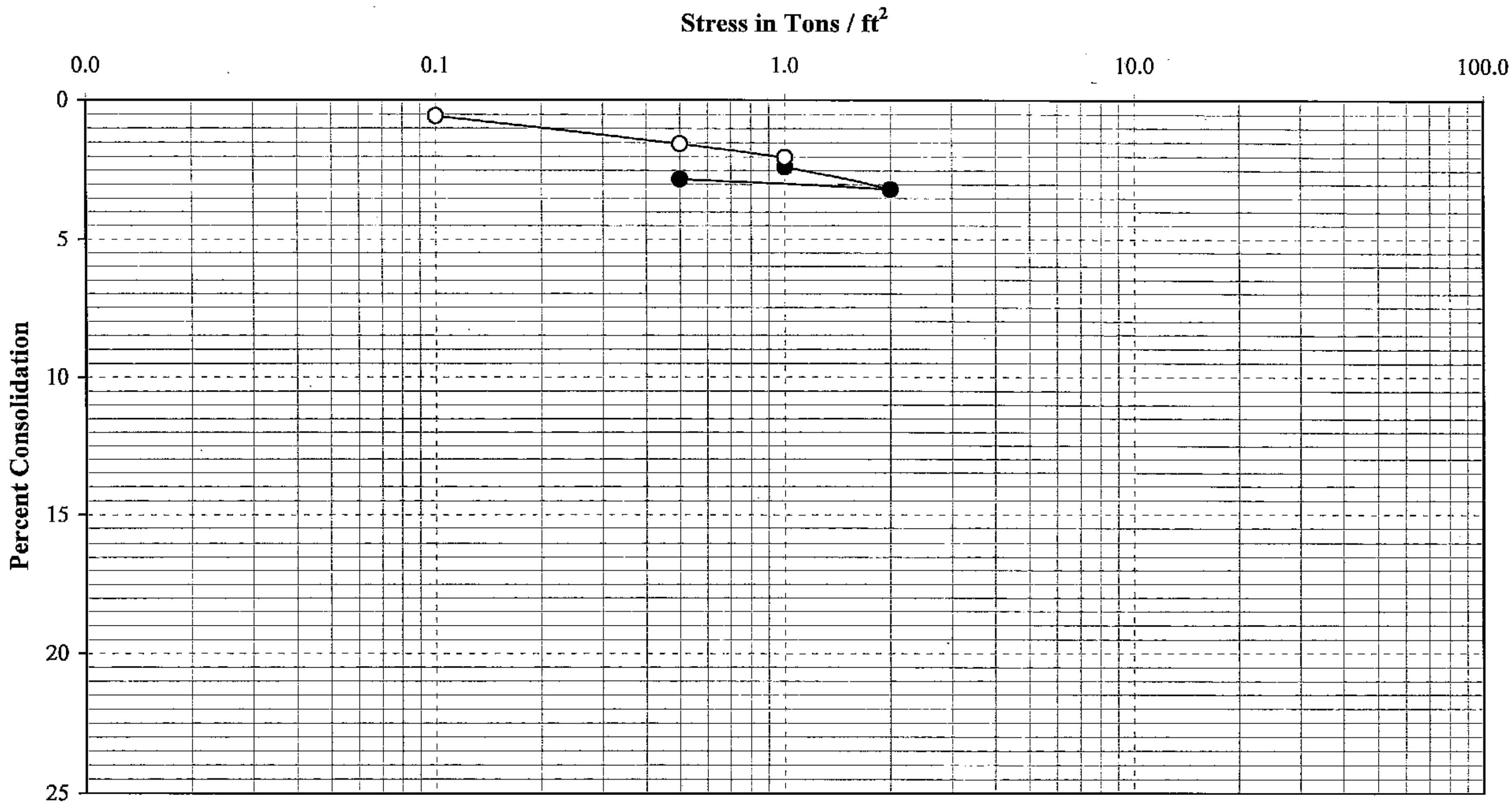
P.N. No: CYG-06-4508

Consolidation Test

Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
B-1	5	10	16	1.0	2.4

Classification : Brown clayey sandy silt (Qa)

Hydroconsolidation = 0.3 %



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Vladi Tomalevski

Date : 05-2006

P.N. No: CYG-06-4508

Consolidation Test

Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
B-1	11	9	19	1.0	2.4

Classification : Brown gravelly silty sand (Qa)

Hydroconsolidation = 0.3 %

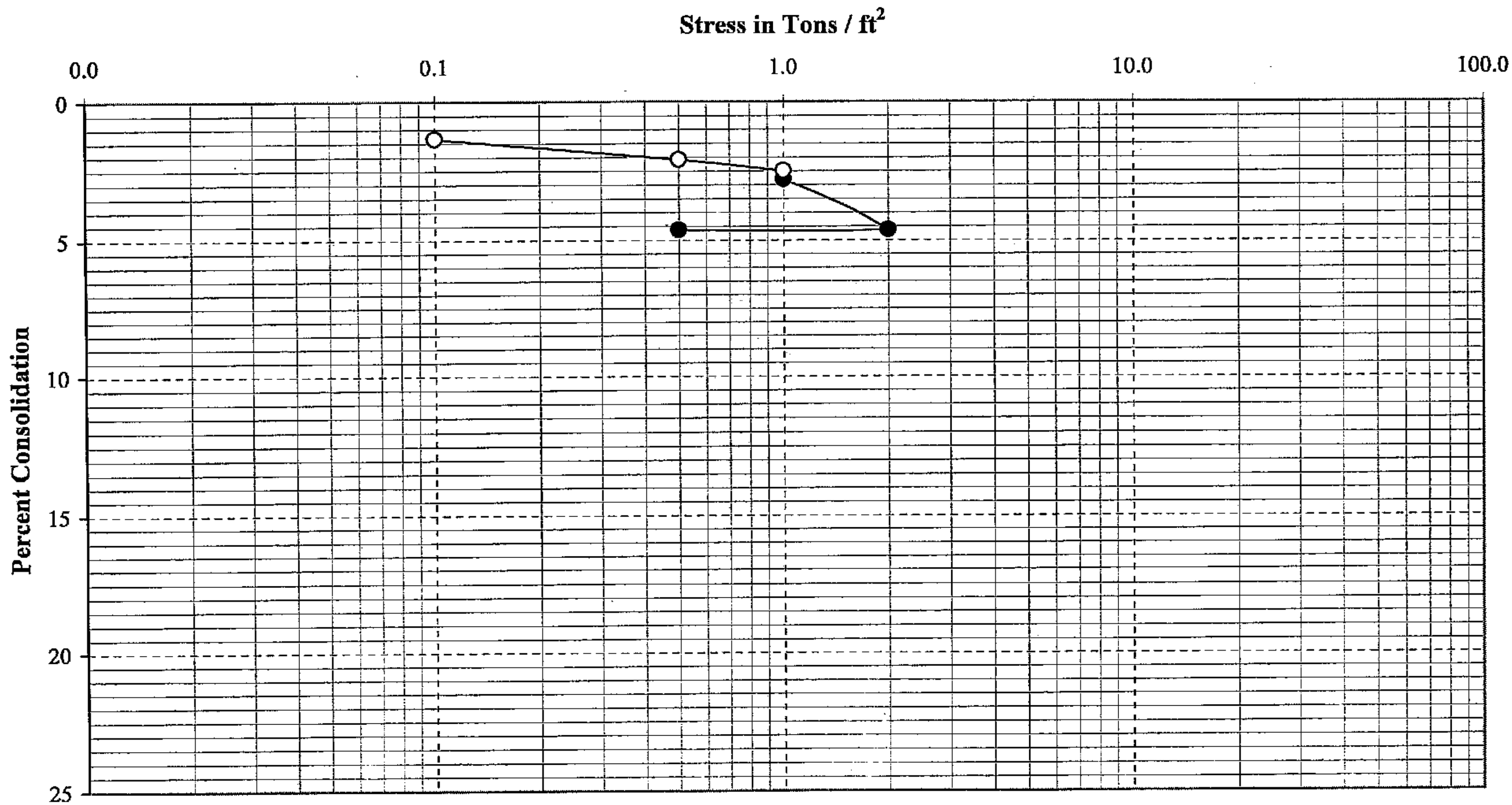

Plate CS - 2

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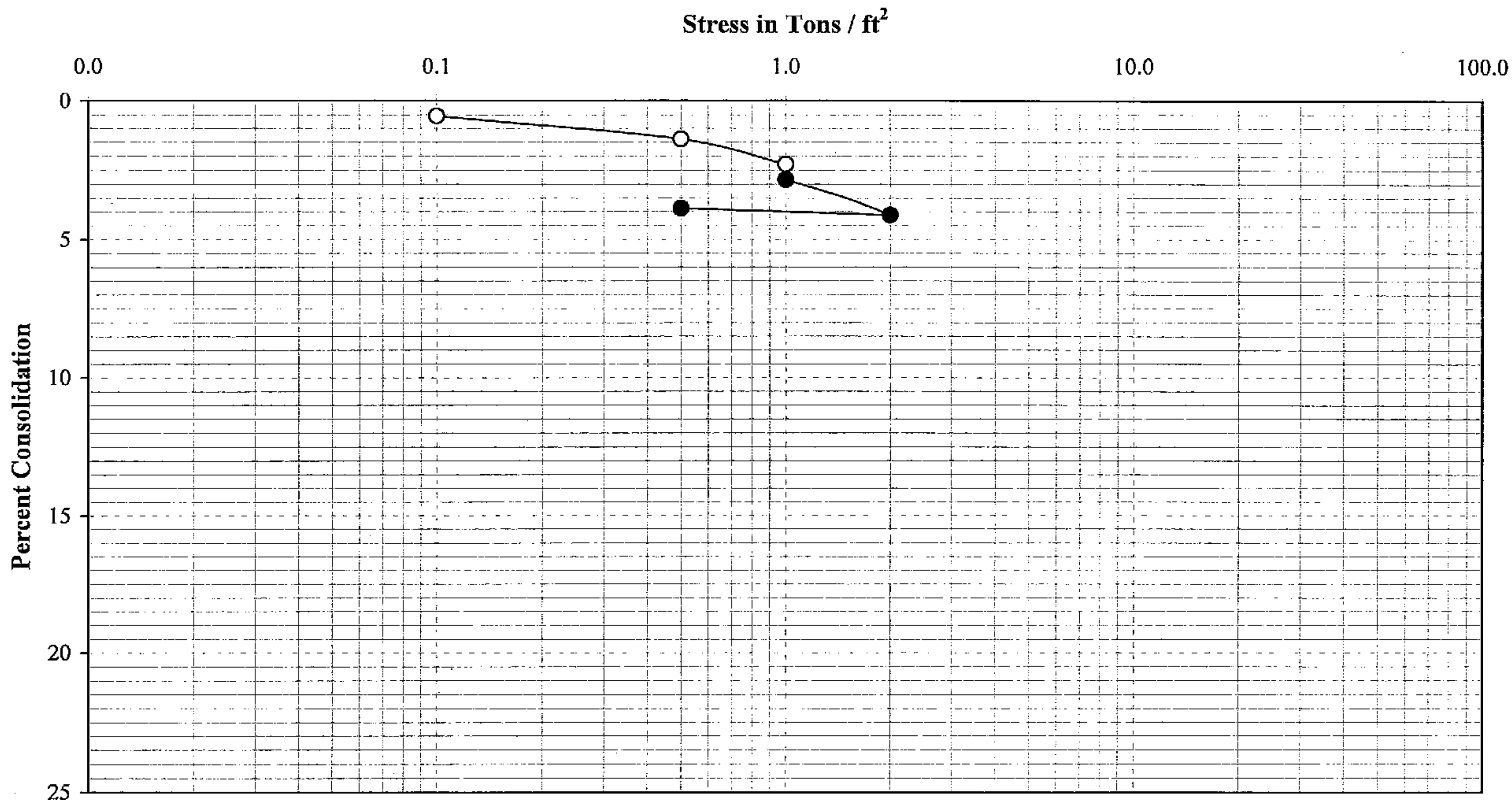
Consolidation Test

Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
B-1	17	6.6	21	1.0	2.4

Classification : Grayish brown gravelly sand(Qa)

Hydroconsolidation = 0.6 %



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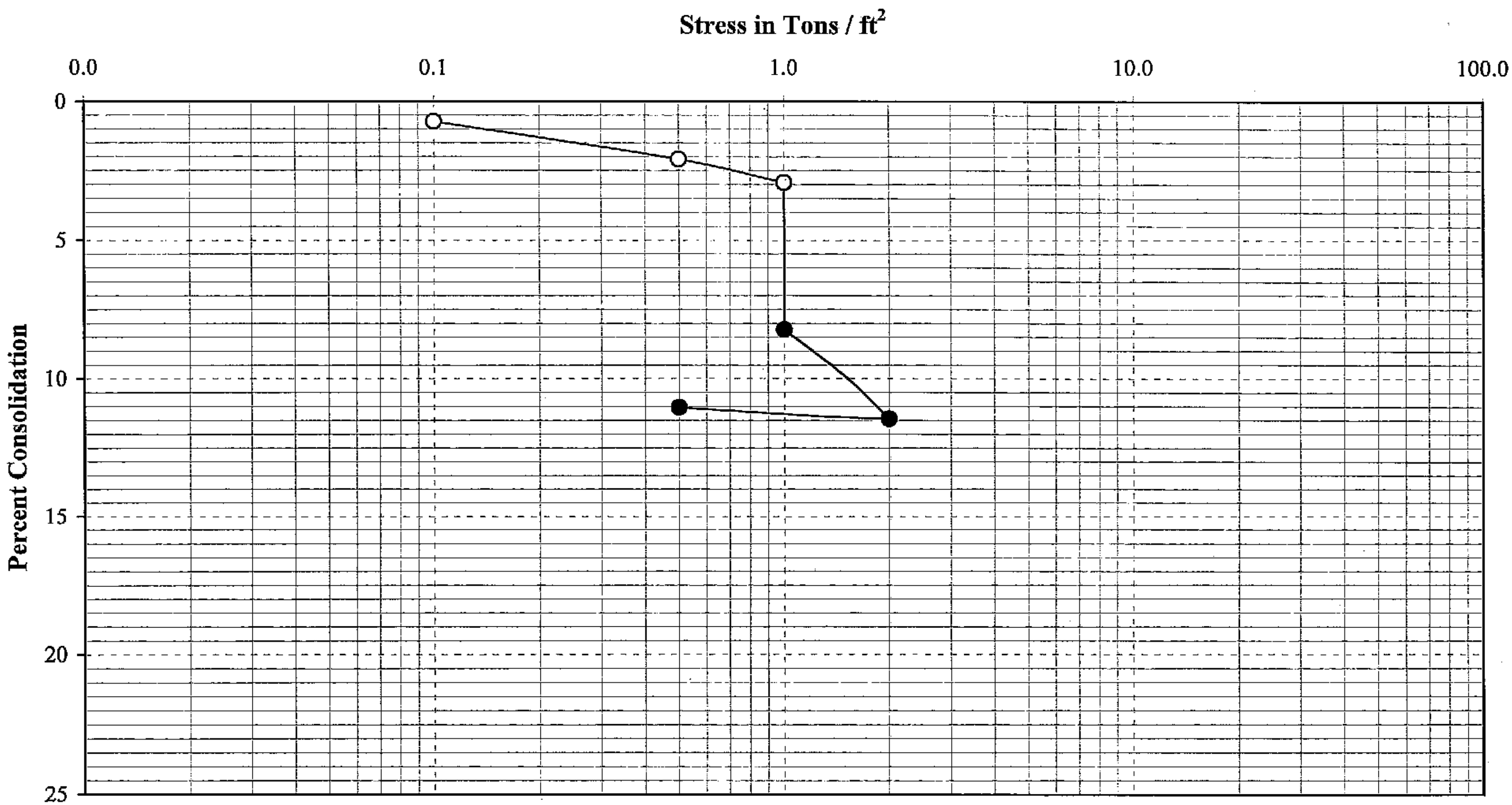
Consolidation Test

Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
B-2	1	15.3	28	1.0	2.4

Classification : Dark brown clayey sandy silt with rootlets (Qa)

Hydroconsolidation = 5.3 %



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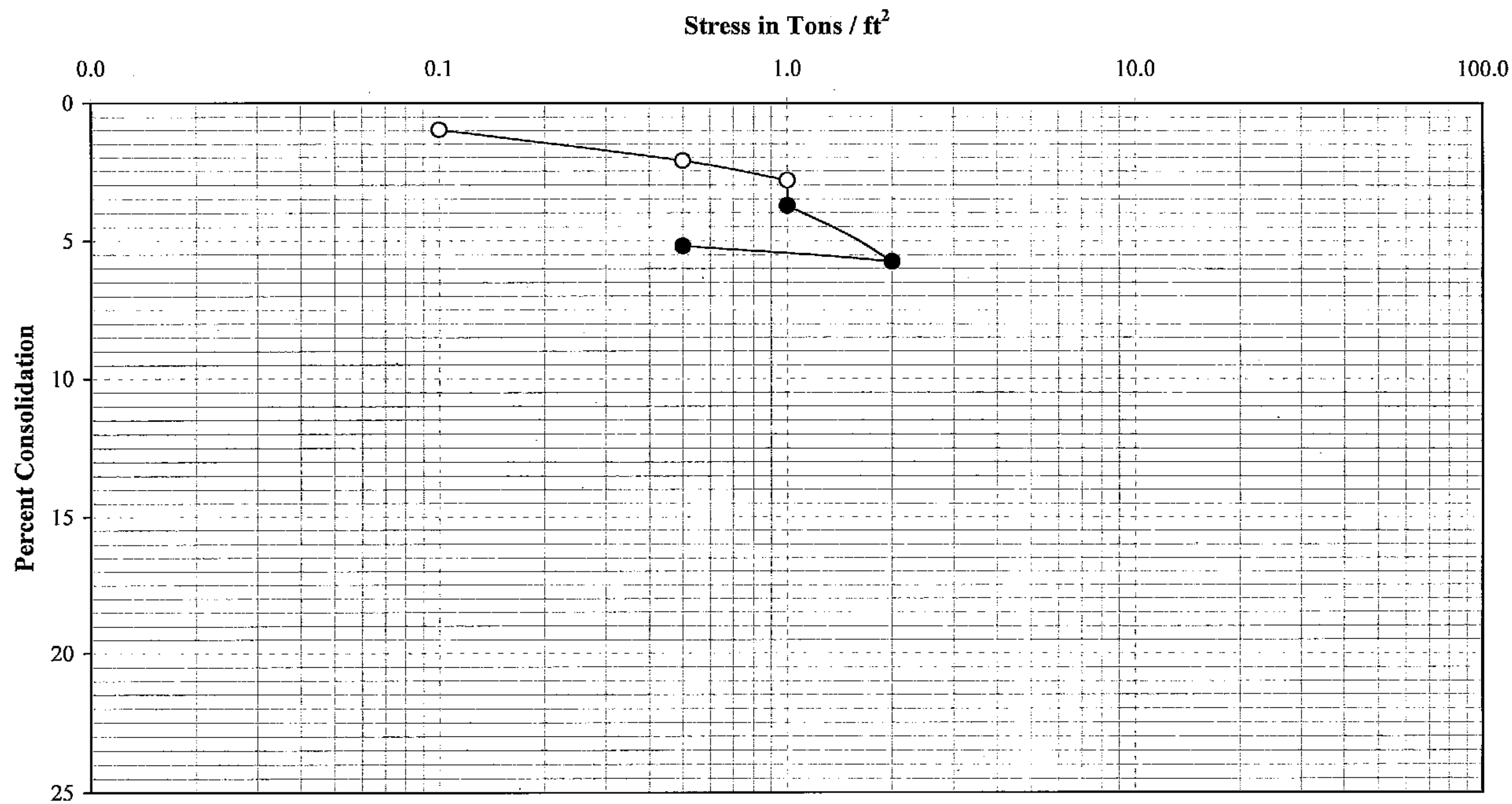
Consolidation Test

Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
B-2	7	15.6	24	1.0	2.4

Classification : Brown gravelly clayey sand (Qa)

Hydroconsolidation = 0.9 %



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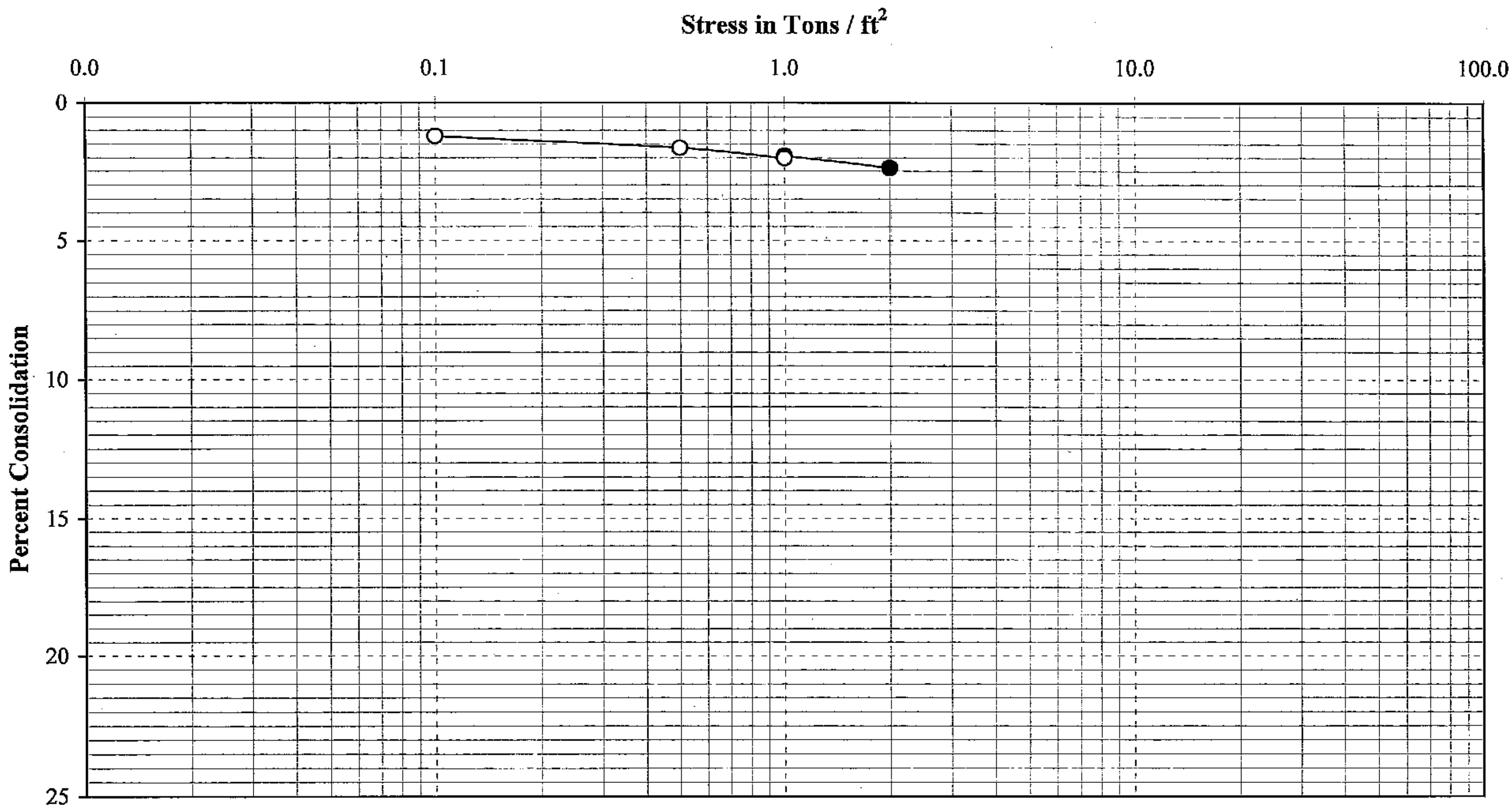
Consolidation Test

Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
B-3	3	14	16	1.0	2.4

Classification : Brown clayey sandy silt with rock fragments (Qa)

Hydroconsolidation = 0 %



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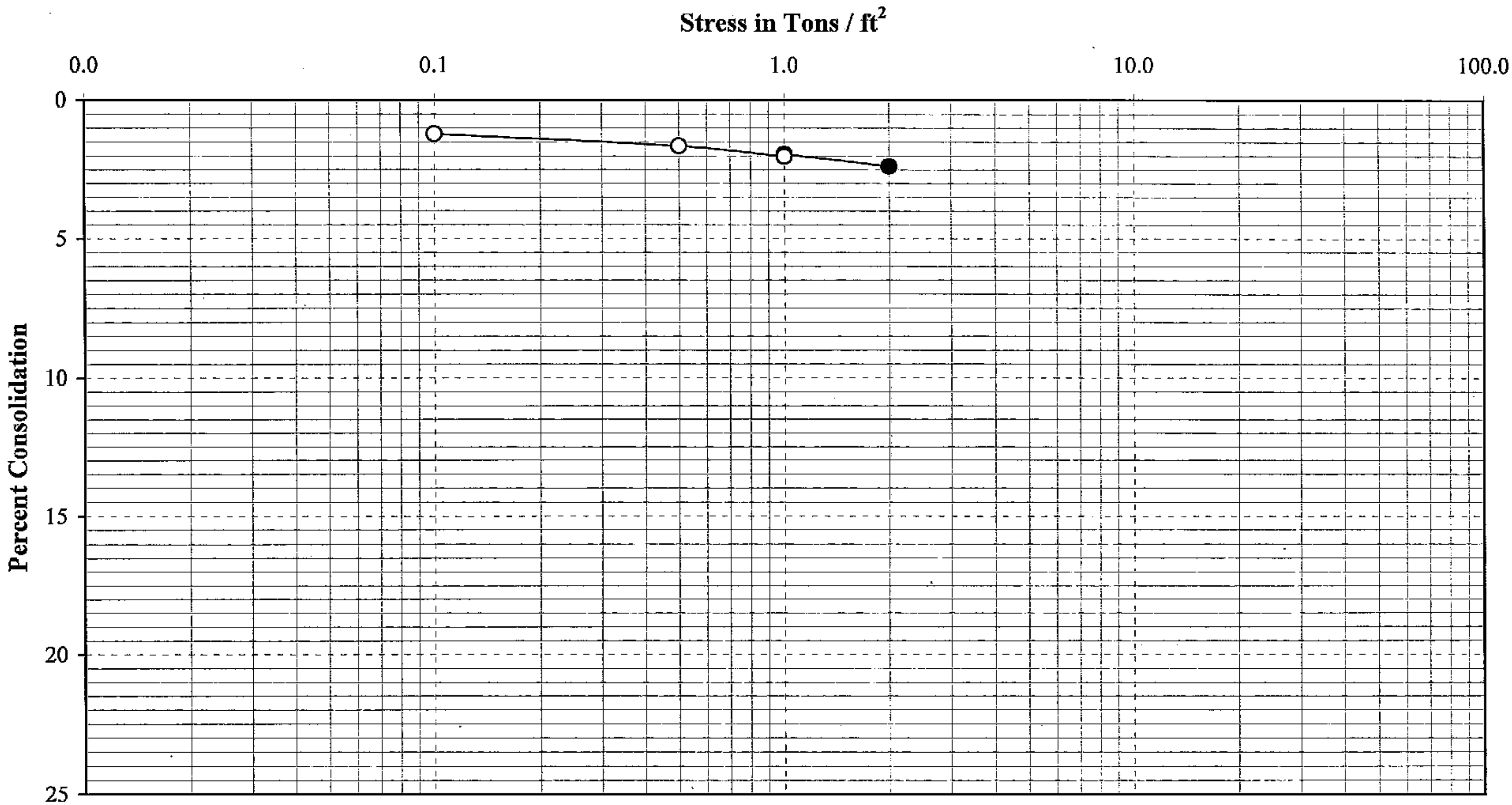
Date : 05-2006	P.N. No: CYG-06-4508

Consolidation Test

Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
B-3	9	16	18	1.0	2.4

Classification : Brown sandy silt (Qa)

Hydroconsolidation = 0 %



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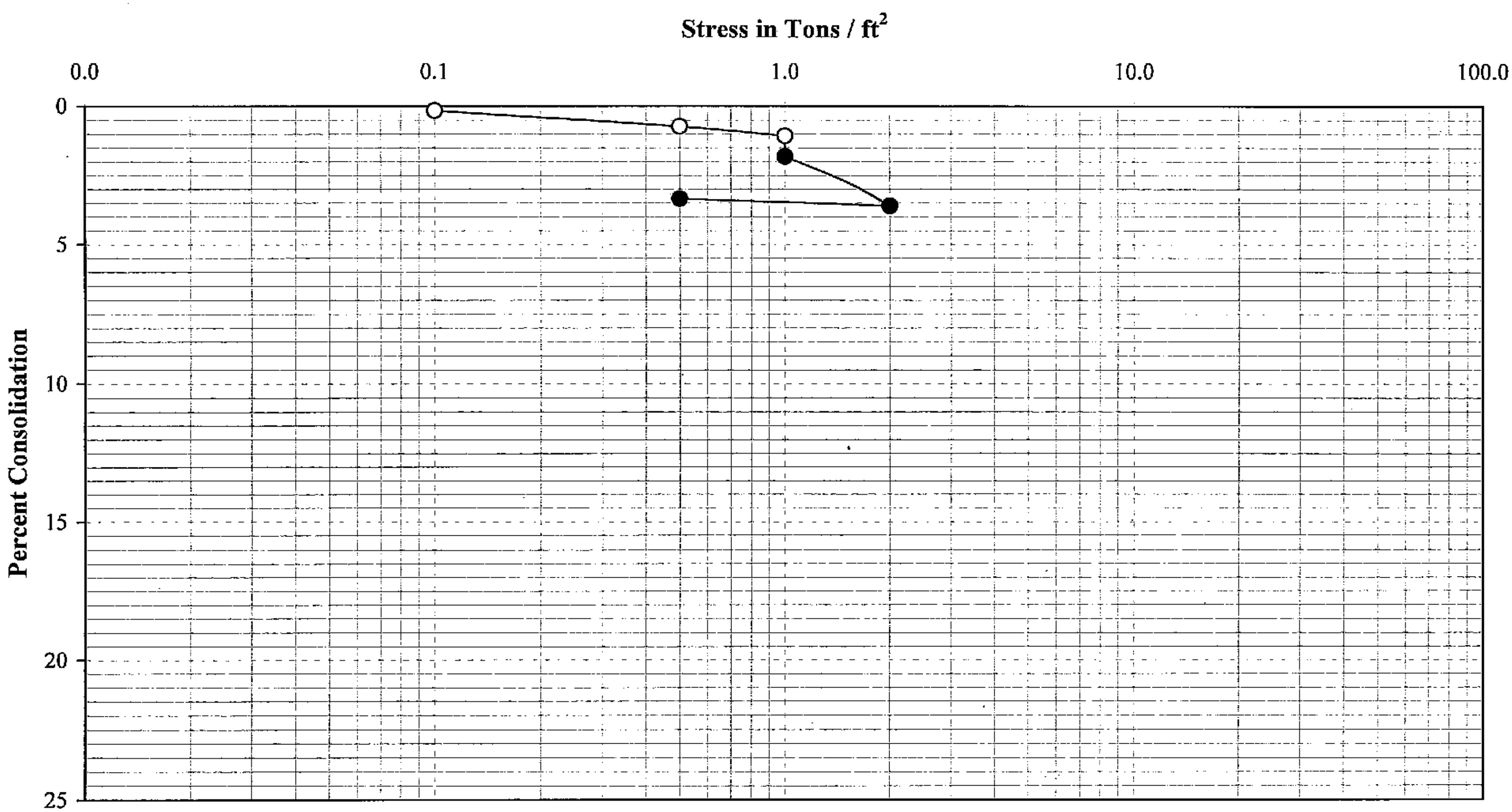
Date : 05-2006 | P.N. No: CYG-06-4508

Consolidation Test

Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
B-4	13	8	16	1.0	2.4

Classification : Grayish brown gravelly sand(Qa)

Hydroconsolidation = 0.8 %



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Vladi Tomalevski

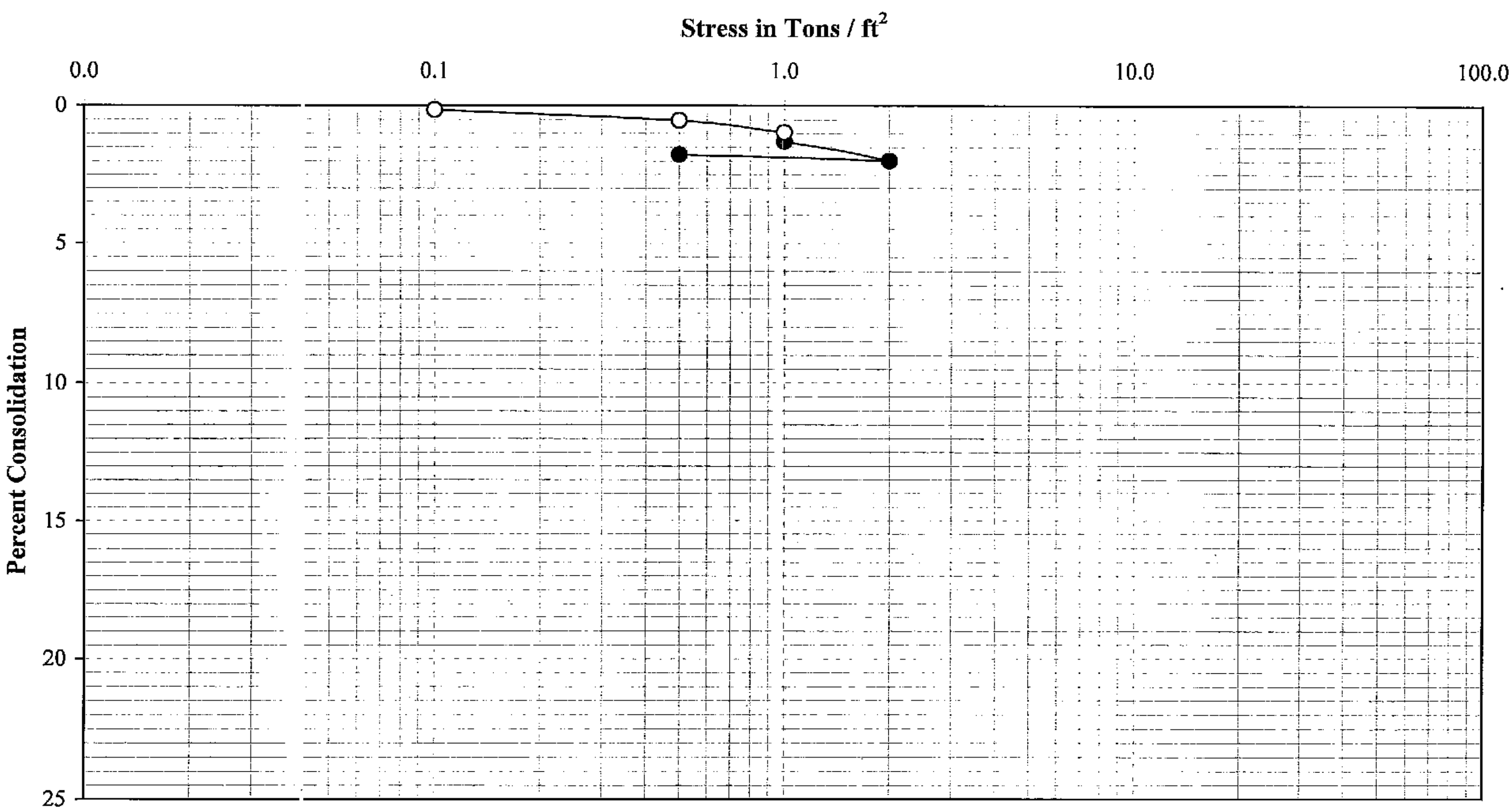
Date : 05-2006	P.N. No: CYG-06-4508

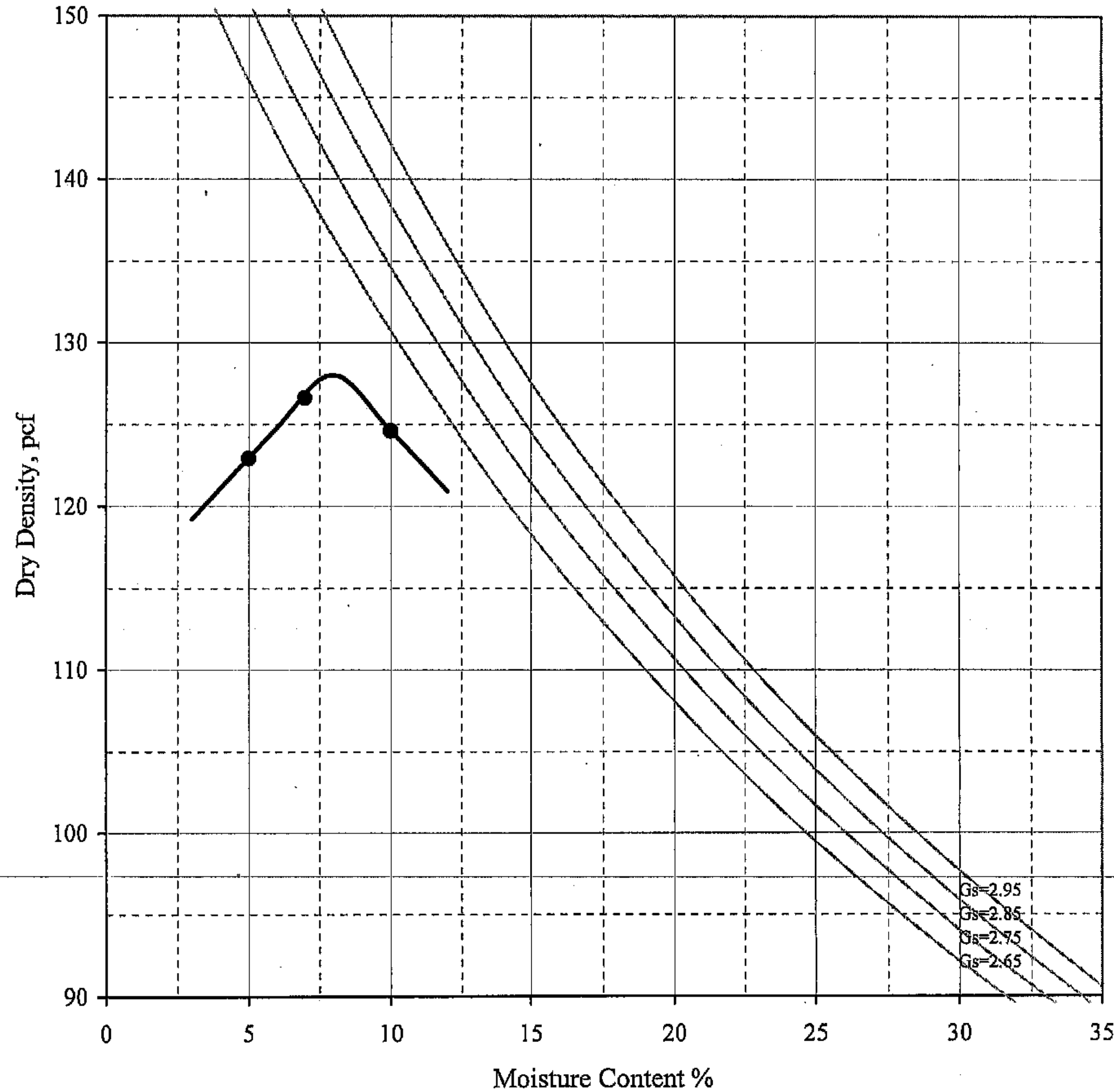
Consolidation Test

Boring	Depth (feet)	Water Content (%) Before	Water Content (%) After	Height (inches)	Diameter (inches)
B-4	15	10	13	1.0	2.4

Classification : Grayish brown gravelly sand (Qa)

Hydroconsolidation = 0.3 %





Maximum Dry Density = 128 pcf Optimum Moisture Content = 8 %	Boring : B-1 Depth : 0 - 5' Description : Brown clayey sandy silt (Qa)
C. Y. GEOTECH, INC. Geotechnical Engineering and Engineering Geology	Vladi Tomalevski

Date : 05-2006	P.N. No.: CYG-06-4508

Compaction Curve