PRELIMINARY GEOTECHNICAL EVALUATION
PROPOSED VALLEY CENTER VIEW PROPERTIES RETAIL
APN 188-231-34, VALLEY CENTER
SAN DIEGO COUNTY, CALIFORNIA

508-013

FOR

VALLEY CENTER VIEW PROPERTIES, L.P.
3936 HORTENSIA STREET
SAN DIEGO, CALIFORNIA 92110

W.O. 5654-A2-SC FEBRUARY 27, 2009

GeoSoils, Inc.

Geotechnical • Geologic • Coastal • Environmental

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February 27, 2009

W.O. 5654-A2-SC

Valley Center View Properties, L.P.
3936 Hortensia Street
San Diego, California 92110

Attention: Mr. Jerry Gaughan

Subject: Preliminary Geotechnical Evaluation, Proposed Valley Center View Properties Retail, APN 188-231-34, Valley Center, San Diego County, California

Dear Mr. Gaughan:

In accordance with your request and authorization, this report presents the results of GeoSoils, Inc.'s (GSI) preliminary geotechnical evaluation of the subject property. The purpose of this study was to evaluate the onsite soils and geologic conditions and their effects on the proposed site development from a geotechnical viewpoint. In general, our study was to preliminarily evaluate potential remedial removal depths, evaluate the potential for adverse geologic conditions, develop preliminary recommendations for earthwork construction, and to provide preliminary geotechnical recommendations for foundations and retaining walls, and pavement design for the proposed commercial development.

EXECUTIVE SUMMARY

Based on our review of data (see Appendix A), field exploration, laboratory testing, and geologic and engineering analyses, the site appears suitable for the proposed commercial development, from a geotechnical viewpoint, provided the recommendations presented in the text of this report are properly implemented. The most significant elements of this study are summarized below:

- The site is predominantly mantled by localized undocumented artificial fill and Quaternary-age colluvium which in turn, are underlain by dense to very dense, Cretaceous-age granitic bedrock of the southern California Batholith. All undocumented fill, Quaternary-age colluvium, and highly weathered granitic bedrock are considered compressible under loading and therefore, unsuitable for the support of engineered fill and/or settlement-sensitive improvements in their existing state. At this time, remedial removal depths are anticipated to be about 1 to

4 feet below the existing grades. However, localized deeper remedial removals cannot be precluded and should be anticipated. Overexcavation to 2 feet below the lowest foundation is recommended to provide uniformity.

- Regional and perched groundwater were not encountered during our field investigation. Regional groundwater is not anticipated to adversely affect the proposed development provided that the recommendations in this report are properly implemented. In general, perched groundwater conditions, along zones of contrasting permeabilities (i.e., fill/bedrock contacts and bedrock discontinuities) may not be precluded from occurring in the future due to site irrigation, poor drainage conditions, or damaged utilities, and should be anticipated. Mitigation of perched water conditions may include, but not necessarily be limited to: subdrainage and/or more onerous foundation design and slab treatment. This should be disclosed to all interested/affected parties.

- Substructures (i.e., underground tanks) should be designed for the effects of hydrostatic pressure from post-development perched groundwater.

- Our review of the preliminary grading plans prepared by Aquaterra Engineering, Inc. ([AEI], 2008) indicates maximum planned excavations (i.e., cuts) on the order of 39 feet. Based on field observations and our experience with similar projects, GSI anticipates that some of the granitic corestones in proposed cut areas may require hard rock blasting so that they may be reduced for transport and fill placement. GSI also anticipates that the decomposed granite matrix, surrounding the corestones, will likely be rippable with a Caterpillar D-9 bulldozer equipped with a single shank ripping tooth. However, trenching for underground utilities and foundations may be problematic when using standardized trenching equipment. Therefore, GSI recommends that building pad areas be overexcavated at least 2 feet below the lowest foundation element for foundation and under-slab utility construction, and to provide uniformity. In addition, driveway/parking areas should be overexcavated at least 1 foot below the lowest underground utility invert to facilitate trenching for such. A rock hardness/rippability analysis was not a part of GSI's authorized scope of work.

- Laboratory testing indicates that site soils are generally very low in expansion potential (Expansion Index [E.I.] = 0 to 20) with a plasticity index less than 15. Conventional foundation systems may likely be used for this soil condition on a preliminary basis. Actual foundation design and construction recommendations will be provided at the conclusion of grading once laboratory testing of finish grade soils has been completed.

- Preliminary laboratory testing, regarding saturated resistivity, soil pH, and soluble sulfates, and chlorides indicates that site soils are moderately corrosive to ferrous metals when saturated, moderately alkaline with respect to soil acidity/alkalinity, present a negligible sulfate exposure to concrete, and are below the threshold limit for chloride exposure. Consultation with a qualified corrosion engineer is recommended regarding piping, steel reinforcement, concrete, etc.

- Based upon the dense to very dense nature of the Cretaceous-age granitic bedrock that underlies the site at relatively shallow depths, and in light of the recommended remedial earthwork, the potential for liquefaction and associated adverse effects to impact site development is considered very low.

- Assuming proper care, maintenance, and normal rainfall, the planned graded slopes are considered to be grossly and surficially stable provided that they are constructed in accordance with the recommendations in this report.

- Site drainage should be evaluated by the project civil consultant to prevent ponding of water and down-gradient scour.

- Budgetary provisions for appropriate mitigation of the above should be included in project planning.

- The recommendations presented in this report should be incorporated into the design and construction considerations of the project.

The opportunity to be of service is sincerely appreciated. If you should have any questions, please do not hesitate to contact any of the undersigned.

Respectfully submitted,

GeoSoils, Inc.

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Project Geologist

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SCOPE OF SERVICES

The scope of our services has included the following:

1. Review of available soils and geologic reports, for the site area (see Appendix A).

2. Geologic site reconnaissance and mapping.

3. Subsurface exploration consisting of four exploratory test pit excavations and two supplemental hand-auger borings (see Appendix B).

4. General areal seismicity evaluation (see Appendix C).

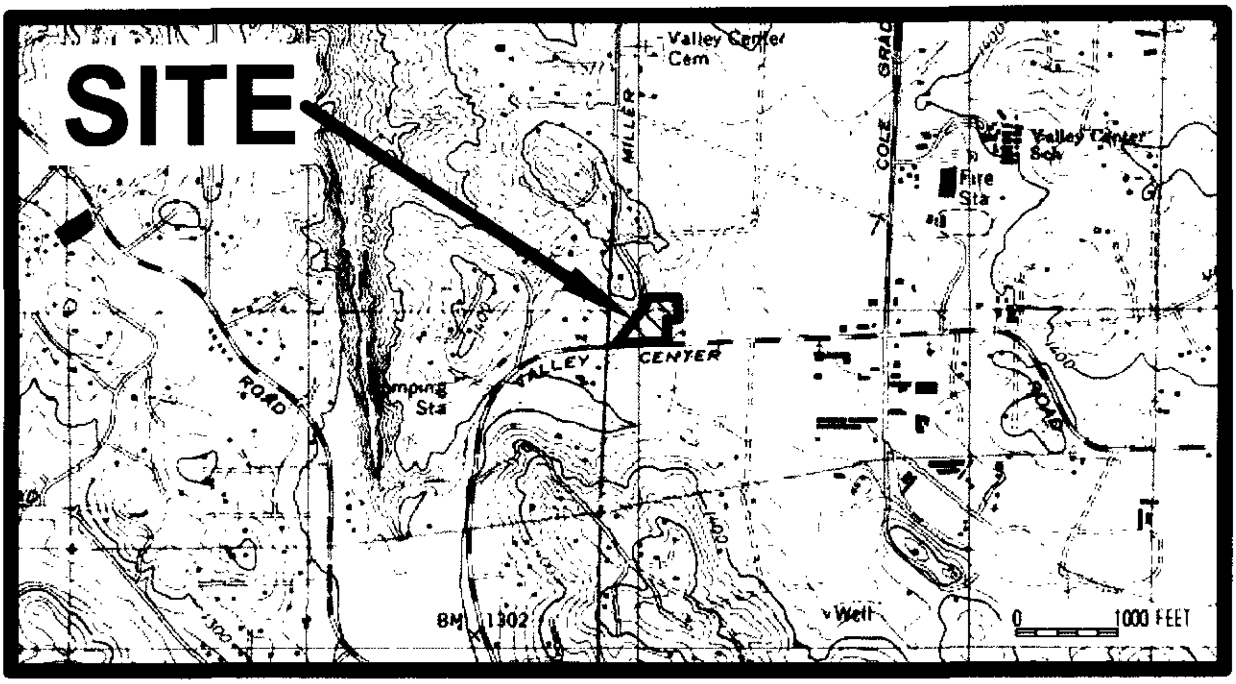
5. Pertinent laboratory testing of representative soil samples collected during our subsurface exploration program (see Appendix D).

6. Appropriate engineering and geologic analysis of data collected.

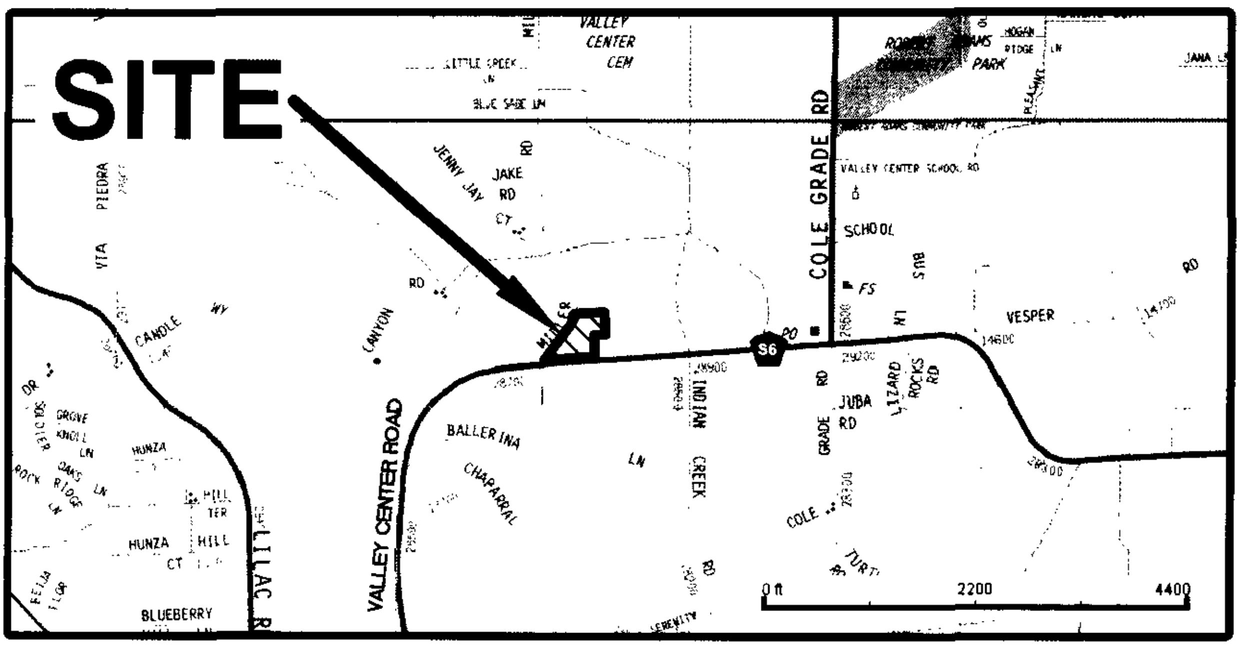
7. Preparation of this preliminary geotechnical report, including test pit and boring logs, laboratory test results, regional seismic data, and presenting general earthwork factors, recommendations for site grading, and preliminary foundation retaining wall, and pavement design.

SITE CONDITIONS AND PROPOSED DEVELOPMENT

The subject site consists of an irregularly shaped, vacant parcel in the Community of Valley Center, San Diego County, California (see Figure 1, Site Location Map). The site is bounded by Valley Center Road to the south, Miller Road to the west, relatively open space to the north, and open space as well as, existing commercial development to the east. Topographically, the site is generally flat-lying to moderately sloping with general slope gradients ranging between 2:1 and 4:1 (horizontal:vertical [h:v]), or flatter. However, steeper slope gradients occur locally. In general, these slopes descend toward all quadrants except north. According to Aquaterra Engineering, Inc. (AEI), site elevations range between approximately 1,385 feet Mean Sea Level (MSL) to about 1,314 feet MSL. Site drainage appears to be accommodated via sheet flow along flatter areas and concentrated flows in the natural drainages. In general, site drainage is directed toward all quadrants except north, and ultimately toward the south. Vegetation consists of grasses, weeds, and low-growth shrubs. Numerous bedrock outcrops occur throughout the site.



Base Map: National Geographic, 2004, TOPO! Digital Map Software, U.S.G.S. Valley Center Quadrangle, California -- San Diego Co., 7.5 Minute, dated 1996, current 1999.



Base Map: The Thomas Guide, San Diego County, Street Guide and Directory, 2008 Edition, by Thomas Bros. Maps, pages 1070 and 1090.

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SITE LOCATION MAP

Figure 1

Based upon a review of the preliminary grading plan by AEI (2008), it is our understanding that proposed development includes preparing the site for the construction of two (2) one-to two-story office buildings and one (1) one-story commercial building (gas station) with associated driveway, parking, underground utility, and onsite storm water treatment improvements. The realignment of Miller Road is also proposed but the realignment appears to have been recently completed by Caltrans during the widening of Valley Center Road. Cut and fill grading appears necessary to achieve the design grades with maximum cuts and fills on the order of 39 and 12 feet, respectively. Maximum height or 2:1 (horizontal:vertical [h:v]), or flatter, and cut and fill slopes on the order of 22 and 5 feet are shown. The referenced plan also indicates that some grade differentials will be accommodated by retaining walls, with maximum heights of about 10 feet.

GSI anticipates that the proposed structures would likely consist of wood-frame and masonry block construction with continuous spread and isolated footings and concrete slab-on-grade floors. Building loads are assumed to be typical for this type of relatively light commercial construction. It is our understanding that sewage would be collected in a holding tank and then be pumped to a disposal field located north of the site. GSI further understands that the proposed septic design has been previously evaluated by others.

FIELD INVESTIGATION

An initial field study occurred on March 5 and 6, 2008 in conjunction with field investigations for the adjacent parcels. That work consisted of geologic reconnaissance mapping, and the excavation of four exploratory test pits with a rubber-tire backhoe. A supplemental field exploration was performed January 19, 2009 to observe the current site conditions and advance two hand-auger borings. The test pit excavations and borings were logged by a geologist from our firm. Representative bulk and undisturbed samples were collected for appropriate laboratory testing. Test Pit and Boring Logs are presented in Appendix B. The approximate locations of the test pits and borings are shown on Plate 1 (Geotechnical Map) which uses AEI (2008) as a base.

GEOLOGY

Regional Geologic Setting

The subject property is located within a prominent natural geomorphic province in southwestern California known as the Peninsular Ranges (Weber, 1977). It is characterized by steep, elongated mountain ranges and valleys that trend northwesterly. The mountain ranges are underlain by basement rocks consisting of pre-Cretaceous metasedimentary rocks, Jurassic metavolcanic rocks, and Cretaceous granitic rocks of the southern California batholith.

Bedrock in the region has been faulted and fractured by both strike-slip and compressional northwest-trending faults, which are related to the San Andreas transform-fault system. Some of these fault zones have remained active to the present time, including the San Andreas fault zone, the San Jacinto fault zone, the Elsinore fault zone, and the Newport-Inglewood - Rose Canyon fault zone. No known active faults have been mapped within the site (Bryant and Hart, 2007; Kennedy, 1999).

Site Earth Materials

The geologic units, encountered within the project site, consist of undocumented artificial fill, Quaternary-age colluvium, and Cretaceous-age granitic bedrock of the southern California batholith. The earth materials are generally described below from youngest to oldest; and their limits, based on the available data, are indicated on Plate 1.

Undocumented Artificial Fill (Map Symbol - Afu)

Undocumented artificial fill was encountered as stockpiled earth materials along the northern portion of the site as well as a recently graded area near the southern end of the site. Here, the undocumented fill appears to be associated with Caltrans' recent widening of Valley Center Road. The undocumented fill generally consists of brown to dark brown, dry to moist, loose to medium dense silty sand. Stockpiles of granitic bedrock cobbles and boulders, and asphaltic concrete were also present, locally. All undocumented fill is potentially compressible in its present state. As, such these materials will likely require mitigation in the form of removal and recompaction, if located below a 1:1 (h:v) projection from the limit of any planned fill and/or the bottom outboard edge of any proposed settlement-sensitive improvement. All oversized undocumented fill materials greater than 12 inches in any dimension will require special handling and placement if used for structural fills. Owing to environmental concerns, asphaltic concrete should not be used in any structural fill.

Quaternary-age Colluvium (Not Mapped)

Quaternary-age colluvium was encountered at the surface in Test Pits TP-1 through TP-3 and Hand Auger HA-1. These soils generally consist of dark brown, brown, and grayish brown silty sand that was dry to wet and loose to medium dense. Locally, the colluvium contained trace to abundant angular to sub-angular pebble- to cobble-sized granitic clasts. The Quaternary-age colluvium is generally considered unsuitable for the support of engineered fill and/or settlement-sensitive improvements in its existing state. Therefore, these soils should be removed and reused as properly compacted fill.

Cretaceous-age Granitic Bedrock (Map Symbol - Kgr)

Cretaceous-age granitic bedrock (also referred to as granitics) occurs at the surface and underlies the undocumented fill and colluvium throughout the site. As observed in Test

Pits TP-1 and TP-2, the upper +1½ to 2½ feet of these earth materials is weathered and consists of brown sandy clays, clayey sands, sandy silts, and silty sands that were generally moist and stiff/medium dense. Where unweathered, the bedrock generally consists of dry to moist, medium dense to very dense, brownish gray to gray (quartz diorite and tonalite composition) that broke to silty sand upon excavation. Where unweathered, the bedrock is considered suitable for the support of engineered fill and/or settlement-sensitive improvements. The weathered portions should be removed and reused as properly compacted fill.

GEOLOGIC STRUCTURE

Bedrock joints were observed generally striking between N32°E and N75°W with moderate to steep (52° to 90°) northerly and southerly dips. A bedrock shear, likely originating from batholith emplacement, was observed striking N24°W and dipping 69° northeast. Based on the planned 2:1 (h:v) cut slopes shown on AEI (2008) and the inclination of observed bedrock joints, GSI does not anticipate that bedrock structure will adversely affect cut slope stability, on a preliminary basis. However, all cut slopes should be mapped by a geologist during grading. In addition, all retaining wall backcuts should be observed by a geologist prior workers entering backcut excavations.

GROUNDWATER

Regional groundwater was not encountered within our test pits and borings at the site. However, perched water conditions were observed at the surface and in test pits located east and south (offsite) of the subject site during our 2008 field exploration. The observed perched water is likely the result of the infiltration (percolation) of seasonal rainwater through the loose/soft surficial soils and its collection, or ponding, along the contact with underlying, relatively impermeable substrata (i.e., granitic bedrock). Based on the planned grades shown on AEI (2008), regional groundwater is not anticipated to adversely affect the proposed development, provided that the recommendations in this report are properly implemented. Perched water conditions, developing along zones of contrasting permeabilities (i.e., fill/bedrock contacts and bedrock discontinuities), may not be precluded from occurring in the future due to site irrigation, poor drainage conditions, or damaged utilities, and should be anticipated. Mitigation of perched water conditions may include, but not necessarily be limited to: subdrainage and/or more onerous foundation design and slab treatment. This should be disclosed to all interested/affected parties.

ROCK HARDNESS EVALUATION

AEI (2008) indicates planned excavations up to 39 feet deep. Although, analyzing the excavation characteristics of the granitic bedrock was not a part of our authorized scope of work, GSI provides the following opinions regarding the facilitation of the planned excavations based on our experience with similar sites.

In general, the granitic bedrock at the site consists of very dense, resistant corestones nested in a decomposed granite matrix. The planned excavations into the bedrock will likely require that the individual corestones be dislodged from the surrounding matrix. During this process, there is potential that corestones will be encountered that are too large to move with standard grading equipment. In these instances, hard rock blasting will likely be required to create smaller rock constituents for transport and fill placement.

GSI does not anticipate that the decomposed granite matrix will present significant difficulty and/or refusal during the planned excavations provided that a Caterpillar D-9 bulldozer with a single-shank ripping tooth. However, the potential for localized blasting cannot be entirely precluded.

The bedrock will likely present significant difficulty and/or refusal during trenching for foundations and underground utilities. During our initial field exploration, excavations into bedrock with a rubber-tire backhoe varied from moderately to very difficult with practical refusal encountered in Test Pits TP-3 and TP-4 at approximately 3½ and 1½ feet below the existing grade, respectively. Thus, overexcavating building pads to at least 2 feet below the lowest foundation to facilitate foundation trenching and uniformity is recommended during grading. In addition, the client should consider overexcavating driveway and parking areas to at least 1 foot below the lowest underground utility invert elevation to facilitate trenching for underground utilities, storage tanks, etc. Overexcavation for rock hardness is not a geotechnical requirement but should be included in cost vs. benefit comparison analyses for project budgetary provisions.

FAULTING AND REGIONAL SEISMICITY

Faulting

The site is situated in an area of active, as well as potentially active, faults. Our review indicates that there are no known active faults crossing the site within the areas proposed for development, and the site is not located within an earthquake fault zone (Bryant and Hart, 2007; Kennedy, 1999). However, there are a number of faults in the southern California area that are considered active and would have an effect on the site in the form of moderate to strong ground shaking, should they be the source of an earthquake. These include, but are not limited to: the San Andreas fault; the San Jacinto fault; the Elsinore fault; the Coronado Bank fault zone; and the Newport-Inglewood/Rose Canyon fault zone.

The location of these and other major faults relative to the site are indicated on Figure 2 (California Fault Map). The possibility of ground acceleration or shaking at the site may be considered as approximately similar to the southern California region as a whole. The major faults and fault zones in southern California that could have a significant effect on the site should they experience significant activity are listed in Appendix C. Based upon its proximity to the site, the Julian segment of the Elsinore fault (M_w = 7.1 [California Geological Survey, 2002]) is considered the design earthquake fault for this site.

The acceleration-attenuation relation of Bozorgnia, Campbell, and Niazi (1999) has been incorporated into EQFAULT (Blake, 2000a). For this study, peak horizontal ground accelerations anticipated at the site were determined based on the random mean plus 1 sigma attenuation curve. EQFAULT is a computer program by Thomas F. Blake (2000a), which performs deterministic seismic hazard analyses using up to 150 digitized California faults as earthquake sources.

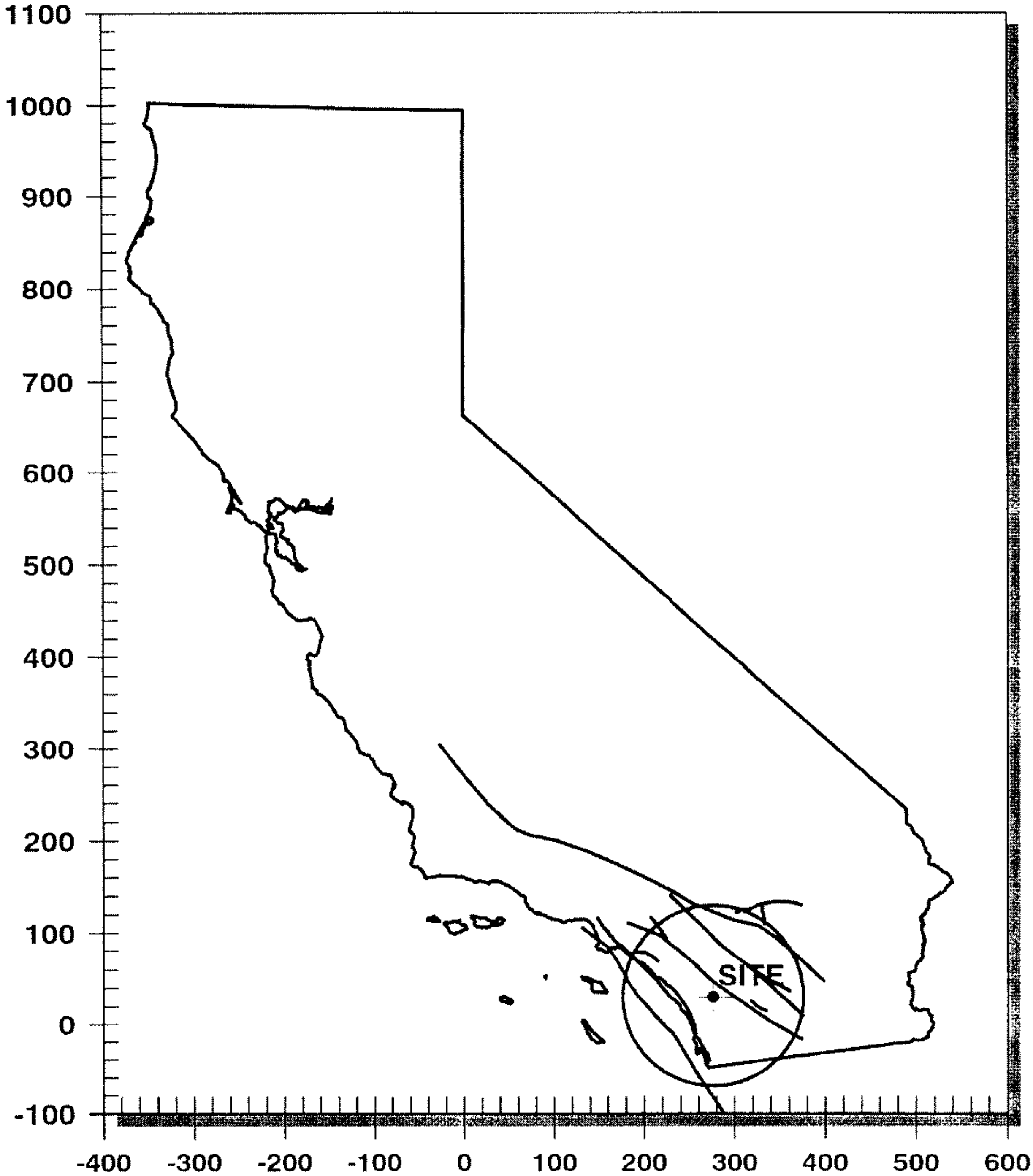
The program estimates the closest distance between each fault and a given site. If a fault is found to be within a user-selected radius, the program estimates peak horizontal ground acceleration that may occur at the site from an upper bound ("maximum credible") earthquake on that fault. Site acceleration (g) is computed by one of many user-selected acceleration-attenuation relations that are contained in EQFAULT. Based on the EQFAULT program, the peak horizontal ground acceleration from an upper bound event at the site may be on the order of 0.40g.

Historical site seismicity was evaluated with the acceleration-attenuation relations of Bozorgnia, Campbell, and Niazi (1999) and the computer program EQSEARCH (Blake, 2000b). This program performs a search of the historical earthquake records for magnitude 5.0 to 9.0 seismic events within a 100-kilometer radius, between the years 1800 to June 2008. Based on the selected acceleration-attenuation relationship, a peak horizontal ground acceleration is estimated, which may have effected the site during the specific event listed. Based on the available data and the attenuation relationship used, the estimated maximum (peak) site acceleration during the period 1800 to June 2008 was 0.11g.

A probabilistic seismic hazards analyses was performed using FRISKSP (Blake, 2000c) which models earthquake sources as 3-D planes and evaluates the site specific probabilities of exceedance for given peak acceleration levels or pseudo-relative velocity levels. Based on a review of this data, and considering the relative seismic activity of the southern California region, a peak horizontal site acceleration (PHSA) on the order of 0.22g was determined for the site. This value was chosen as it corresponds to a 10 percent probability of exceedance in 50 years (or a 475-year return period).

CALIFORNIA FAULT MAP

VALLEY CENTER VIEW PROPERTIES, L.P.



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CALIFORNIA FAULT MAP

Figure 2

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Seismic Shaking Parameters

The table below summarizes the site-specific design criteria obtained from the 2007 CBC (based on the International Building Code [IBC], International Code Council, Inc. [ICCI], 2006), Chapter 16 Structural Design, Section 1613. We used the computer program Seismic Hazard Curves and Uniform Hazard Response Spectra, provided by the United States Geological Survey (USGS, 2008). The short spectral response uses a period of 0.2 seconds.

IBC SEISMIC DESIGN PARAMETERS		
PARAMETER	**VALUE**	**IBC-06 REFERENCE**
Site Class	C	Table 1613.5.2
Spectral Response - (0.2 sec), S_s	1.40g	Figure 1613.5(3)
Spectral Response - (1 sec), S_1	0.53g	Figure 1613.5(4)
Site Coefficient, F_a	1.0	Table 1613.5.3(1)
Site Coefficient, F_v	1.3	Table 1613.5.3(2)
Maximum Considered Earthquake Spectral Response Acceleration (0.2 sec), S_{MS}	1.40g	Section 1613.5.3 (Eqn 16-37)
Maximum Considered Earthquake Spectral Response Acceleration (1 sec), S_{M1}	0.69g	Section 1613.5.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (0.2 sec), S_{DS}	0.94g	Section 1613.5.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.46g	Section 1613.5.4 (Eqn 16-40)

Conformance to the criteria above for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur in the event of a large earthquake. The primary goal of seismic design is to protect life, not to eliminate all damage, since such design may be economically prohibitive. Cumulative effects of seismic events are not included in the code, and regular maintenance and repair following significant seismic events (i.e., $\geq M_w 4.5$) will likely be necessary.

Seismic Hazards

The following list includes other seismic related hazards that have been considered during our evaluation of the site. The hazards listed are considered negligible and/or mitigated as a result of site location, soil characteristics, and typical site development procedures:

- Liquefaction Potential
- Seismic Settlement
- Surface Fault Rupture
- Ground Lurching or Shallow Ground Rupture
- Seiche
- Tsunami

It is important to keep in perspective that in the event of a maximum probable or upper bound earthquake occurring on any of the nearby major faults, strong ground shaking would occur in the subject site's general area. Potential damage to any structure(s) would likely be greatest from the vibrations and impelling force caused by the inertia of a structure's mass than from those induced by the hazards considered above. This potential would be no greater than that for other existing structures and improvements in the immediate vicinity.

LIQUEFACTION EVALUATION

Liquefaction describes a phenomenon in which cyclic stresses, produced by earthquake induced ground motion, create excess pore pressures in relatively cohesionless soils. These soils may thereby acquire a high degree of mobility, which can lead to lateral movement/sliding, consolidation and settlement of loose sediments, sand boils, and other damaging deformations. This phenomenon occurs only below the water table, but after liquefaction has developed, it can propagate upward into overlying, non-saturated soil, as excess pore water dissipates. Typically, liquefaction has a relatively low potential at depths greater than 50 feet and is unlikely and/or will produce vertical strains well below 1 percent for depths below 60 feet when relative densities are 40 to 60 percent and effective overburden pressures are two or more atmospheres (i.e., 4,000 psf [Seed, 2005]).

Liquefaction susceptibility is related to numerous factors and the following concurrent conditions must exist, or have the potential to exist, for liquefaction to occur: 1) sediments must be relatively young in age and not have developed a large amount of cementation; 2) sediments must consist mainly of medium- to fine- grained, relatively cohesionless sands; 3) the sediments must have low relative density; 4) free groundwater must be present in the sediment; and, 5) the site must have a potential for a design seismic event of a sufficient duration and large enough magnitude, to induce straining of soil particles. Our evaluation indicates that at least two or three of the five concurrent conditions do not exist at the site as evidenced by the occurrence of dense to very dense, Cretaceous-age granitic bedrock underlying the site at relatively shallow depths. Furthermore, the recommended remedial earthwork includes removing and replacing the low density surficial soils with properly compacted fill. Therefore, the potential for liquefaction and its associated adverse effects to impact site development is considered very low.

OTHER GEOLOGIC HAZARDS

Mass Wasting

Mass wasting refers to the various processes by which earth materials are moved downslope in response to the force of gravity. No evidence of significant mass wasting was observed during our field exploration at the site. Based on the relatively flat-lying to moderately sloping, surrounding terrain and the dense nature of the underlying granitic bedrock, the potential for mass wasting to affect the site is low.

Rockfalls

Based on our review of AEI (2008), it appears that the granitic outcrops, within the site, will be removed during earthwork construction. In addition, should a granitic outcrop become dislodged from an offsite slope, the relatively gentle gradients of the surrounding slopes would not create velocities strong enough for offsite rocks to damage proposed site improvements.

Subsidence

Subsidence is a phenomenon whereby a lowering of the ground surface occurs as a result of a number of processes. These include dynamic loading during grading, fault activity or fault creep as well as groundwater withdrawal.

Ground subsidence is expected to occur over the site in exposed areas prior to filling, due to equipment working (vibrations) and the effect of loading of additional fill (weight). Both of these factors are variable and difficult to estimate.

Ground subsidence (consolidation) due to vibrations would depend on the equipment being used, the weight of the equipment, repetition of use and the dynamic effects of the equipment. Most of these factors cannot be evaluated and may be beyond ordinary estimating possibilities.

Our evaluation indicates that ground subsidence at the site would be primarily due to vibrations and loading, prior to filling, and may be on the order of 1 to 2 inches in some areas. Subsidence should generally occur during grading. In general, field subsidence would depend upon equipment, haul routes, duration of grading, and the effect of dynamic loading.

SLOPE STABILITY EVALUATION

Based upon our review of the preliminary grading plan by AEI (2008), maximum height, planned cut and fill slopes are respectively on the order of 22 and 5 feet and inclined at gradients of 2:1 (h:v), or flatter. Based on the presence of moderate to high angle bedrock joints and assuming proper care, maintenance, and normal rainfall, the planned graded slopes are preliminarily considered grossly and surficially stable provided that the recommendations in this report are properly incorporated into project design and construction. All cut slopes should be mapped and further evaluated by a geologist during construction.

LABORATORY TESTING

Classification

Soils were visually classified in the field according to the Unified Soil Classification System. Classifications were supplemented by Particle-Size Analyses testing in general accordance with ASTM D 422-63 performed on a representative sample. The soil classifications are shown on the Test Pit and Boring Logs (see Appendix B).

Laboratory Standard

The maximum dry density and optimum moisture content was evaluated in general accordance with ASTM D-1557 for a representative, composite sample of the near-surface soils encountered in Hand Auger HA-1. The moisture-density relationship obtained for this soil is shown below:

LOCATION AND DEPTH (FEET)	SOIL TYPE	MAXIMUM DRY DENSITY (PCF)	OPTIMUM MOISTURE CONTENT (%)
HA-1 @ 0-4½	Dark Yellow Brown, CLAYEY SAND	132.0	9.0

Expansion Potential

Expansion testing was performed on a representative sample of the near-surface soils in general accordance with ASTM 4829. The results of expansion testing are presented in the following table.

LOCATION AND DEPTH (FT)	EXPANSION INDEX	EXPANSION POTENTIAL
HA-1@ 0-4½	<5	Very Low

Direct Shear Testing

Shear testing was performed on a remolded, composite sample of the near-surface soils in general accordance with ASTM test method D-3080. The sample was remolded to 90 percent relative compaction at optimum moisture content. Test results are presented in the following table and in Appendix D.

LOCATION AND DEPTH (FEET)	PRIMARY		RESIDUAL	
	COHESION (PSF)	FRICTION ANGLE (DEGREES)	COHESION (PSF)	FRICTION ANGLE (DEGREES)
HA-1 @ 0-4½ (remolded)	303	33	202	33

Particle-Size Analysis

An evaluation was performed on selected representative soil samples in general accordance with ASTM D 422-63. Particle size analyses were performed on selected samples from our exploratory boring. The grain-size distribution curves are presented in Appendix D. These test results were utilized in evaluating the soil classifications in accordance with the Unified Soil Classification System.

Sulfate/Corrosion Testing

A representative sample of the site soil was analyzed for saturated resistivity, soil pH, and soluble sulfates. Preliminary results indicate that site soils are moderately alkaline (pH=8.1) with respect to soil acidity/alkalinity, and are moderately corrosive to ferrous metals when saturated (saturated resistivity = 7,500 ohm-cm). Moderately corrosive soils are considered to range between 2,000 and 10,000 ohm-cm (Caltrans, 1999; Romanoff, 1989).

Testing also indicates that the percentage by weight soluble sulfates (SO_4^{2-}) in the tested sample is non-detectable. Thus, site soils are considered to have a negligible sulfate exposure to concrete on a preliminary basis. In addition, testing indicates that the percentage by weight soluble chlorides (Cl^{1-}) in the tested sample is non-detectable. Thus, site soils are considered to be below the threshold limit for chloride exposure on a preliminary basis. Alternative testing methods and additional comments may be obtained from a qualified corrosion engineer regarding exposed metal, foundations, piping, etc., as needed. Upon completion of grading, additional testing of soils (including import materials) for corrosion to concrete and metals should be performed prior to the construction of utilities and foundations. Preliminary test results are presented in Appendix D.

Resistance Value (R-value) Testing

R-value testing was performed in accordance with the latest revisions to the Department of Transportation, State of California, Material & Research Test Method No. 301. Testing indicates a R-value of 77. R-value test results are presented in Appendix D.

EMBANKMENT FACTORS

Embankment factors (shrinkage) for the site have been estimated based upon our experience with other sites in the general vicinity. It is apparent that shrinkage would vary with depth and with areal extent over the site, based on previous site use. Variables include vegetation, weed control, discing, and previous filling or exporting. However, all these factors are difficult to define in a three-dimensional fashion. Therefore, the information presented below represents average shrinkage and bulking values, using the following assumptions.

Undocumented Fill	10 - 15% shrinkage
Colluvium.	10 - 15% shrinkage
Weathered Granitic Bedrock.	0 - 8% shrinkage to 0 - 2% bulking
Granitic Bedrock (Excavated)	5% shrinkage to 10% Bulk
Granitic Bedrock (Shot)	15% to 20% Bulk

An additional shrinkage factor item would include the removal of individual large plants or trees. The root structures/root balls of these plants and trees vary in size; but when pulled, they may result in a loss of ½ to 1½ cubic yards. This factor needs to be multiplied by the number of significant plants or trees present to evaluate the net loss. Further, the degree of compaction achieved by the grading contractor may also affect shrinkage and bulking, by up to about 5 percent. The above factors indicate that earthwork balance of the site would be difficult to define and flexibility in design is essential to achieve a balanced end product.

PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

General

Based on our field exploration, laboratory testing and geotechnical engineering analyses, it is our opinion that the site appears suitable for the proposed development from a geotechnical engineering and geologic viewpoint, provided that the recommendations presented in the following sections are incorporated into the design and construction phases of site development. However, it will be necessary to review the final grading plan (precise and/or rough grade) when it becomes available to evaluate if any revisions to the recommendations presented herein are needed. The primary geotechnical concerns with respect to the proposed development are:

- Depth to competent bearing materials.
- Perched water and effects on proposed development, grading procedures/fill placement, and potential for perched water to occur both during grading and after development.
- Substructures and hydrostatic pressure.
- Expansion and corrosion potential of site soils over the life of the project.
- Rock Hardness
- Regional seismic activity.

The recommendations presented herein consider these as well as other aspects of the site. The engineering analyses performed concerning site preparation and the recommendations presented herein have been completed using the information provided and obtained during our field work.

In the event that any significant changes are made to proposed site development, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and the recommendations of this report evaluated or modified in writing by this office. Foundation design parameters are considered preliminary until the foundation design, layout, and structural loads are provided to this office for review.

1. Soil engineering, observation, and testing services should be provided during grading to aid the contractor in removing unsuitable soils and in his effort to compact the fill.

2. Geologic observations should be performed during grading to further evaluate geologic conditions. Although unlikely, if adverse geologic structures are encountered, supplemental recommendations and earthwork may be warranted.

3. Removals should consist of all undocumented fill, Quaternary-age colluvium, and weathered granitic bedrock. Our subsurface exploration indicates that removal depths should be on the order of 1 to 4 feet below the existing grades. However localized deeper remedial removals cannot be precluded and should be anticipated.

4. Regional and perched groundwater are not anticipated to adversely affect site development. However, perched groundwater conditions, along zones of contrasting permeabilities (i.e., fill/bedrock contacts and bedrock discontinuities), should not be precluded from occurring in the future due to site irrigation, poor drainage conditions, or damaged utilities, and should be anticipated. Mitigation of perched water conditions may include, but not necessarily be limited to: subdrainage and/or more onerous foundation design. This should be disclosed to all interested/affected parties.

5. Our laboratory test results generally indicate that soils with very low expansion potentials and with plasticity indices less than 15 underlie the site. Thus, conventional foundation may be used at the site on a preliminary basis. Conventional foundation design and construction recommendations are provided herein.

6. Provided that the recommendations contained herein are properly implemented during design and construction, and that graded slopes are properly maintained over their lifetime, the planned slopes currently shown on AEI (2008) are considered grossly and surficially stable, assuming normal rainfall. All cut slopes should be observed by a geologist during grading. If highly weathered zones or adverse geologic structure are exposed within the cut slopes supplemental recommendations will be provided in the field. Such recommendations may include the use of stabilization fills or inclining the slope flatter than the inclination of the adverse geologic structure.

7. Owing to the potential for perched water, substructures (i.e. underground tanks) that cannot be drained will need to consider the effects of hydrostatic pressure, uplift, etc.

8. The seismicity-acceleration values provided herein should be considered during the design of the proposed development.

General Grading

All grading should conform to the guidelines presented in Appendix J of the California Building Code ([CBC], California Building Standards Commission [CBSC], 2007), the County of San Diego, and Appendix E (this report), except where specifically superceded in the text of this report. When code references are not equivalent, the more stringent code should be followed. During earthwork construction, all site preparation and the general grading procedures of the contractor should be observed and the fill selectively tested by a representative(s) of GSI. If unusual or unexpected conditions are exposed in the field, they should be reviewed by this office and, if warranted, modified and/or additional recommendations will be offered. All applicable requirements of local and national construction and general industry safety orders, the Occupational Safety and Health Act (OSHA), and the Construction Safety Act should be met.

Demolition/Grubbing

1. Existing structures (if encountered), vegetation, and any miscellaneous debris should be removed from the areas of proposed grading.

2. Cavities or loose soils remaining after site clearance should be cleaned out and observed by the soil engineer. The cavities should be replaced with fill materials

that have been moisture conditioned to at least optimum moisture content and compacted to at least 90 percent of the laboratory standard.

Treatment of Existing Ground

1. Remedial removals should consist of all undocumented fill, Quaternary-age colluvium, and weathered granitic bedrock. Removals should be completed below a 1:1 (h:v) projection from the limit of any proposed structure and/or planned fill. At this time, we anticipate removal depths to be on the order of 1 to 4 feet below the existing grades. However, localized deeper removals cannot be precluded and should be anticipated.

2. Subsequent to the above removals, the upper 12 inches of the exposed subsoils should be scarified, brought to at least optimum moisture content, and recompacted to a minimum relative compaction of 90 percent of the laboratory standard.

3. All undocumented artificial fill, Quaternary-age colluvium, and weathered granitic bedrock may be reused as compacted fill provided that major concentrations of vegetation and miscellaneous debris are removed prior to or during fill placement, and that near optimum moisture conditions are attained.

Overexcavation

In order to provide uniform foundation support, reduce the potential for differential settlement, and facilitate foundation and underground utility trenching, we recommend that at least 2 feet of compacted fill be provided beneath the proposed foundation and 1 foot of compacted fill be placed beneath the lowest underground utility invert. Portions of the building pad requiring less than 2 feet of planned fill, below the bottom of the footing, after remedial removals have been completed should be overexcavated to provide for this minimum requirement. Additionally, the maximum to minimum fill thickness across any building pad should not exceed a ratio of 3:1 (maximum:minimum). Overexcavation should be completed to at least 5 feet outside the outboard edge of the perimeter foundation or below a 1:1 (h:v) projection from the bottom outboard edge of the perimeter foundation (whichever is greater). The overexcavation subgrade should be inclined to drain toward driveway or parking areas. The overexcavation subgrade should also be scarified at least 12 inches, moisture conditioned to at least the soil's optimum moisture content, and be recompacted to at least 90 percent of the laboratory standard, prior to placing compacted fill.

Engineered Fill Placement

Engineered fill should be cleaned of all deleterious debris, moisture conditioned (or dried back) to at least the soil's optimum moisture content, placed in thin (6- to 8-inch) lifts, and mechanically compacted to at least 90 percent of the laboratory standard (ASTM D 1557). The placement of engineered fill should be performed under the observation and testing of the geotechnical engineer. Any proposed import material should be evaluated by the geotechnical engineer prior to importation. At least 3 business days of lead time should be provided to allow for the necessary laboratory testing. Import soils for a fill cap should be very low to low expansive (E.I. less than 50) with a P.I. less than 15. Based upon the type of import used, subdrains at the bottom of the fill cap may be necessary, and subsequently recommended depending on import compatibility with onsite soils. Foundation designs may be altered if import materials have a greater expansion and/or plasticity index value(s) than the onsite materials encountered in this investigation.

Rock Placement Guidelines

GSI anticipates that soils used as fill material for this project may contain some rock. Appropriately, the need for rock disposal, subject to the governing agency approval, may be necessary during grading operations. From a geotechnical standpoint, the depth of any rocks, rock fills, or rock blankets, should be a sufficient distance from finish grade. This depth is generally the same as any overexcavation due to cut-fill transitions in hard rock areas, and generally facilitates the excavation of structural footings and substructures. Should deeper excavations be proposed (i.e., deepened footings, utility trenching, etc.), the developer may consider increasing the hold-down depth of any rock fills to be placed, as appropriate. In addition, some agencies/jurisdictions mandate a specific hold-down depth for oversize materials placed in fills. The hold-down depth, and potential to encounter oversize rock, both within fills, and in occurring in cut or natural areas, would need to be disclosed to all interested/affected parties. If approved by the governing agency, the hold-down depth for oversized rock (i.e., greater than 12 inches in any dimension) in fills on this project is provided as 10 feet.

General

Generally for the purpose of this report, the materials may be described as either 12 inches in any dimension or less, greater than 12 and less than 36 inches in any dimension, and greater than 36 inches in any dimension. These three categories set the basic dimensions for where and how the materials are to be placed.

Materials 12 Inches in Any Dimension or less

Since rock fragments along with the overburden materials are anticipated to be a part of the materials used in the grading of the site, a criteria is needed to facilitate the placement of these materials within guidelines which would be workable during the rough grading, post-grading improvements, and serve as acceptable compacted fill.

1. Fines and rock fragments 12 inches in any dimension or less may be placed as compacted fill cap materials within the building pads, slopes, and street areas as

described below. The rock fragments and fines should be brought to <u>at least</u> optimum moisture content and compacted to a minimum relative compaction of 90 percent of the laboratory standard.

2. The purpose for the 12-inch limit is to allow reasonable sized rock fragments into the fill under selected conditions (optimum moisture or above) surrounded with compacted fines. The 12-inch rock size also allows a greater volume of the rock fragments to be handled during grading, while staying in reasonable limits for later onsite excavation equipment (backhoes and trenchers) to excavate footings and utility line trenches.

3. Fill materials 12 inches or less in any dimension should be placed (but not limited to) within the hold-down depth on proposed fill pads, the upper 4 feet of overexcavated cut areas of cut/fill transition pads, and the entire street right-of-way width, including the proposed overexcavated areas and replacement fill areas, from the depth of the lowest utility (within the street and lot), to subgrade, or to the hold-down depth below finish grade. Overexcavation is discussed later in this report.

Materials Greater Than 12 inches and Less Than 36 Inches in any Dimension

1. During the process of bedrock excavation, a significant amount of rock fragments or constituents larger than 12 inches in any dimension may be generated. These significant amounts of oversized materials, greater than 12 and less than 36 inches in any dimension, may be incorporated into the fills utilizing a series of rock blankets.

2. Each rock blanket should consist of rock fragments of approximately greater than 12 and less than 36 inches in any dimension along with fines generated from the proposed cuts and overburden materials from removal areas. The blankets should be limited to 24 to 36 inches in thickness and should be placed with granular fines which are flooded into and around the rock fragments.

3. Rock blankets should be restricted to areas which are at least 1 foot below the lowest utility invert within the street right-of-way, at least the hold-down depth below finish grade on the proposed fill lots, and a <u>minimum</u> of 20 horizontal feet from the face of fill slopes, and outside of any utility laterals.

4. Compaction may be achieved by utilizing wheel rolling methods with scrapers and water trucks, track-walking by bulldozers, and sheepsfoot tampers.

5. Each rock blanket should be completed with its surface compacted prior to placement of any subsequent rock blanket or rock windrow.

6. Minor amounts of rock material in this size range may also be placed a rock windrows (see below).

Materials Greater Than 36 Inches in any Dimension

1. Oversize rock greater than 36 inches in any dimension should be placed in single rock windrows. The windrows should be at least 15 feet or an equipment width apart, whichever is greatest.

2. The void spaces between rocks in windrows should be filled with the more granular soils by flooding them into place.

3. A minimum vertical distance of 3 feet between soil fill and rock lift should be maintained. Also, the windrows should be staggered from lift to lift. Rock windrows should <u>not</u> be placed closer than 20 feet from the face of fill slopes, and outside of any utility laterals.

4. Larger rocks too difficult to be placed into windrows may be individually placed into a dozer trench. Each trench should be excavated into the compacted fill or dense natural ground a minimum of 1 foot deeper than the size of the rock to be buried. After the rocks are placed in the trench (not immediately adjacent to each other), granular fill material should be flooded into the trench to fill the voids.

5. The oversize rock trenches should be no closer together than 15 feet at a particular elevation and <u>at least</u> 20 feet from any fill slope face. Trenches at higher elevations should be staggered and there should be 4 feet of compacted fill between the top of one trench and the bottom of the next higher trench. Placement of rock into these trenches should be under the full-time observation of the geotechnical consultant.

6. Consideration should be given to using oversize materials in open space "green belt" areas that would be designated as non-structural fills.

Substructures Placed in the Hold-Down Depth Zone

Disclosure to all interested/affected parties regarding the proximity of oversize materials, excavation difficulties, etc., that may potentially impact future improvements is recommended. The cap above the hold-down distance is only intended to support shallow foundations of the structure, appurtenant structures, and builder specified improvements. Utility poles, underground storage tanks, or similar improvements that penetrate or nearly penetrate the fill cap should have a site-specific subsurface investigation, and review by the geotechnical consultant, prior to planning, design, and construction.

Graded Slope Considerations and Slope Design

Based upon our review of the preliminary grading plan by AEI (2008), planned cut and fill slopes with maximum respective heights of 22 and 5 feet will be constructed. All graded slopes should be designed and constructed in accordance with the minimum requirements of the County of San Diego, the recommendations in Appendix E, and the following:

- Fill slopes should be designed and constructed at a 2:1 (h:v) gradient, or flatter. Fill slopes should be properly built in accordance with the design standards provided in the CBC (CBSC, 2007), and compacted to a minimum relative compaction of 90 percent throughout, including the slope surfaces. Guidelines for slope construction are presented in Appendix E. Fill slopes will require normal care and maintenance, especially after periods of heavy precipitation.

- Cut slopes should be designed at gradients of 2:1 and should not exceed the heights indicated on AEI (2008), without further analyses. While stabilization of such slopes is not anticipated, locally adverse geologic conditions (e.g., daylighted joints/fractures or severely weathered bedrock) may be encountered which may require remedial grading (i.e., stabilization fills) or laying back of the slope to an angle flatter than the adverse geologic condition.

- Local areas of loose, unconsolidated sandy colluvial materials and/or highly to severely weathered bedrock may be present. Should these materials be exposed in cut slopes, the potential for long term maintenance or possible slope failure exists. Evaluation of cut slopes during grading would be necessary in order to identify any areas of severely weathered bedrock or loose, unconsolidated colluvial soils. Should any of these materials be exposed during construction, the soils engineer/geologist, would assess the magnitude and extent of the materials and their potential affect on long-term maintenance or possible slope failures. Recommendations would then be made at the time of the field inspection.

- Cut slopes as well as retaining wall backcuts should be mapped by the project engineering geologist during grading to allow amendments to the recommendations should exposed conditions warrant alternation of the design or stabilization.

Temporary Slopes

On a preliminary basis, unsupported temporary excavations greater than 4 feet but less than 20 feet deep, should be constructed in accordance with criteria established in Article 6 of the State of California, Construction Safety Orders (Cal-OSHA) for Type "B" soils. Heavy equipment and/or stockpile should not be stored within 'H' feet of any temporary slope (where 'H' equals the height of the temporary slope). Additionally, heavy equipment should not be operated within 'H' feet from the top of any temporary slope (where 'H' equals the height of the temporary slope) . Temporary slopes should be further evaluated by the geotechnical engineer during site grading. Based on the conditions exposed, the possibility of inclining temporary slopes to flatter gradients may be recommended if adverse soil conditions are observed. If the required gradient of any temporary slope conflicts with property boundaries, shoring may be necessary.

PRELIMINARY RECOMMENDATIONS - FOUNDATIONS

General

In the event that the information concerning the proposed development concept is not correct or any changes in the design, location, or loading conditions of the proposed structure(s) are made, the conclusions and recommendations contained in this report are for the current design only and shall not be considered valid unless the changes are reviewed and conclusions of this report are modified or approved in writing by this office.

The information and recommendations presented in this section are considered minimums and are not meant to supercede design(s) by the project structural engineer or civil engineer specializing in structural design. Upon request, GSI could provide additional consultation regarding soil parameters, as related to foundation design. They are considered preliminary recommendations for proposed construction, in consideration of our field investigation, laboratory testing, and engineering analysis.

Based on a conversation with the Client, GSI understands that the proposed commercial buildings will consist of wood-frame and concrete masonry unit (CMU) construction. Although no proposed loading conditions have been provided, GSI anticipates average and maximum static column loads of 50 and 150 kips, respectively. Maximum building perimeter wall loads are anticipated to be on the order of 2 to 5 kips per lineal foot of wall. According to AEI (2008), the finished floor levels are anticipated to be between elevations of about 1,331 and 1,342 feet MSL.

The foundation design and construction recommendations are based on the anticipated loading conditions and laboratory testing and engineering analysis of onsite earth materials by GSI. Conventional-type foundation systems may be used to support the proposed structure(s), provided they are founded in very low expansive, competent bearing material (compacted fill) with a plasticity index less than 15. The proposed foundation systems should be designed and constructed in accordance with the guidelines contained in the CBC (CBSC, 2007) as appropriate. Preliminary recommendations for conventional foundation systems are provided herein. Final foundation recommendations will be provided subsequent to the completion of rough grading and laboratory testing of near-finish grade soils.

Foundation Design - Isolated Spread and Continuous Footings (Very Low Expansion Potential with a P.I. less than 15)

Based on the anticipated foundation loads and laboratory test results, it is our opinion that the proposed structure(s) can favorably be supported by shallow foundations on at least 2 feet of recompacted fill soils overlying the granitic bedrock. Building loads may be supported on continuous or isolated spread footings (typically 18 to 30 inches below planned grades) designed in accordance with the following recommendations.

ALLOWABLE BEARING VALUES FOR FOOTINGS		
DEPTH BELOW LOWEST ADJACENT FINISHED GRADE (INCHES)	ALLOWABLE BEARING CAPACITY FOR INTERIOR SPREAD FOOTINGS (MINIMUM WIDTH = 4 FEET)	ALLOWABLE BEARING CAPACITY FOR CONTINUOUS WALL FOOTINGS (MINIMUM WIDTH = 2 FEET)
18	2.0 ksf	2.0 ksf
24	2.5 ksf	2.5 ksf
30	3.0 ksf	3.0 ksf

The above values are for dead plus live loads and may be increased by one-third for short-term wind or seismic loads. Where column or wall spacings are less than twice the width of the footing, some reduction in bearing capacity may be necessary to compensate for the effects of group action. Reinforcement should be designed in accordance with local codes and structural considerations.

The recommended allowable bearing capacity is generally based on maximum total and differential settlements indicated in this report for building areas. Actual settlement can be estimated on the basis that settlement is roughly proportional to the net contact bearing pressure. The majority of the settlement should occur during construction. Since settlement is a function of footing size and contact bearing pressure, some differential settlement can be expected between adjacent columns or walls where a large differential loading condition exists. However, for most cases, static differential settlements are considered unlikely to those previously indicated. With increased footing depth/width ratios, differential settlement should be less, provided a minimum of 2 feet of compacted fill overlying granitic bedrock is maintained beneath all footings. GSI should review foundation plans and evaluate foundation-specific load patterns. Based upon our review, supplemental recommendations may be necessary.

Other Footing Design Parameters

1. For lateral sliding resistance, a 0.35 coefficient of friction may be utilized for a concrete to soil contact when multiplied by the dead load.

2. Passive earth pressure may be computed as an equivalent fluid having a density of 250 pcf with a maximum earth pressure of 2,500 psf.

3. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third.

4. All footings should maintain a minimum 7-foot horizontal distance between the base of the footing and any adjacent descending slope, and minimally comply with the guidelines depicted on Figure No. 1805.3.1 of the 2007 CBC.

5. Soil generated from footing excavations to be used onsite should be moisture conditioned to at least optimum moisture content and compacted to at least 90 percent minimum relative compaction, whether it is to be placed inside the foundation perimeter or in landscape/right-of-away areas. This material must not alter positive drainage patterns that direct drainage away from the structural area and toward the street.

Construction

The following isolated spread and continuous footing foundation construction recommendations are presented as a minimum criteria from a soils engineering viewpoint. The onsite soil's expansion potential, evaluated by laboratory testing, is generally in the very low (E.I. 0 to 20) range with a P.I. less than 15. Recommendations by the project's design-structural engineer or architect, which may exceed the soils engineer's recommendations, should take precedence over the following minimum requirements. Final foundation design will be provided based on the expansion potential of the near-surface soils encountered during grading.

1. Conventional continuous footings should be founded at a minimum depth of 18 to 30 inches (depending on the allowable bearing value from the previous section) below the lowest adjacent ground surface for typical light commercial building loads. Interior footings may be founded at a minimum depth of 18 to 30 inches below the lowest adjacent ground surface. The entire foundation should be entirely supported by at least 2 feet of compacted fill beneath the footings.

   Footings should have a minimum width of 24 inches. All footings should be reinforced with a minimum of four No. 5 reinforcing bars, two at the top and two No. 5 reinforcing bars at the bottom.

2. Isolated <u>exterior</u> pier and column footings may be constructed 24 inches square by 24 inches deep, and tied to the main foundation in at least one direction with a grade beam. Isolated exterior pier and column footings should be supported by a minimum of 2 feet of compacted fill beneath the footing. Isolated footing reinforcement should be designed by the project structural engineer.

3. A grade beam, reinforced as above and at least 18 inches deep, should be provided across garage, or any other large entrances. The base of the reinforced grade beam should be at the same elevation as the adjoining footings.

4. The slab subgrade moisture content should be at least the soil's optimum moisture content to a depth of 18 to 30 inches below grade.

5. Concrete mix design and slab underlayment recommendations are provided in the "Soil Moisture Considerations" section of this report.

6. Concrete slab-on-grade construction recommendations are provided in the following section.

FLOOR SLAB DESIGN RECOMMENDATIONS - ISOLATED SPREAD AND CONTINUOUS FOOTING FOUNDATION SYSTEMS

General

Concrete slab-on-grade floor construction is anticipated. The following are presented as minimum design parameters for the slab, but they are in no way intended to supercede design by the structural engineer. Design parameters do not account for concentrated loads (e.g., fork lifts, heavy rack loads, other machinery, etc.) and/or the use of freezers or heating boxes.

These recommendations are meant as minimums. The project architect and/or structural engineer should review and verify that the minimum recommendations presented herein are considered adequate with respect to anticipated uses.

Light Load Floor Slabs

The slabs in areas that will receive relatively light live loads (i.e., office space, less than 50 psf) should be a minimum of 5 inches thick and be reinforced with No. 3 reinforcing bar on 18-inch centers in two horizontally perpendicular directions. Reinforcing should be properly supported to ensure placement near the vertical midpoint of the slab. "Hooking" of the reinforcement is not considered an acceptable method of positioning the steel.

The project structural engineer should consider the use of transverse and longitudinal control joints to help control slab cracking due to concrete shrinkage or expansion. Two of the best ways to control this movement are: 1) add a sufficient amount of reinforcing steel to increase the tensile strength of the slab; and 2) provide an adequate amount of control and/or expansion joints to accommodate anticipated concrete shrinkage and expansion. Transverse and longitudinal crack control joints should be spaced no more than 12 feet on center and constructed to a minimum depth of T/4, where "T" equals the slab thickness in inches.

Heavy Load Floor Slabs

The project structural engineer should design the slabs in areas subject to high loads (machinery, forklifts, storage racks, etc.). The Modulus of subgrade reaction (k_S-value) may be used in the design of the floor slab supporting heavy truck traffic, fork lifts, machine foundations, and heavy storage areas. A k_S-value of 175 pounds per square inch per inch (pci) would be prudent to utilize for preliminary slab design. An R-value test and/or plate load test should be used to verify the actual k_S-value on the finish grade soils at the conclusion of grading.

Concrete slabs should be at least 6 inches thick and reinforced with No. 4 reinforcing bars placed 12 inches on center in two horizontally perpendicular directions. Selection of slab thickness compatibility with anticipated loads should be provided by the structural engineer.

Transverse and longitudinal crack control joints should be spaced no more than 14 feet on center and constructed to a minimum depth of T/4. The use of expansion joints in the slab should be considered. Concrete used in slab construction should have a maximum water/cement ratio of 0.5. Spacing of expansion or crack control joints should be modified based on the footprint of the area to be heavily loaded.

Subgrade Preparation

Subgrade material should be compacted to a minimum of 90 percent of the maximum laboratory dry density. Prior to placement of concrete, the subgrade soils should be presoaked to 18 to 30 inches below grade (depending on the footing depth utilized) to at least the soils' optimum moisture content prior to and within 72 hours of the concrete placement. Alternative methods, including sealing the subgrade surface with select sand/base and periodic moisture conditioning, may also be considered, as long as the minimum recommended soil moisture contents are achieved.

POST-TENSIONED SLAB FOUNDATIONS

Post-tensioned slab foundations may be used as an alternative to conventional foundations if higher performance is desired. Recommendations for utilizing post-tensioned slabs on the site are based on soil parameters potentially exposed within the site's near-surface. The recommendations presented below should be followed in addition to those contained in the previous sections, as appropriate. The information and recommendations presented below in this section are not meant to supercede design by a registered structural engineer or civil engineer familiar with post-tensioned slab design. Post-tensioned slabs should be designed using sound engineering practice and be in accordance with local and/or national code requirements.

From a soil expansion/shrinkage standpoint, a common contributing factor to distress of structures using post-tensioned slabs is fluctuation of moisture in soils underlying the perimeter of the slab, compared to the center, causing a "dishing" or "arching" of the slabs. To mitigate this possibility, a combination of soil presaturation and construction of a perimeter cut-off wall should be employed.

Perimeter cut-off walls should be a minimum of 12 inches deep for very low expansive soils. The cut-off walls may be integrated into the slab design or independent of the slab and should be a minimum of 6 inches wide. The concrete slab should be a minimum of 5 inches thick. The actual slab thickness should be determined by the project architect and or structural engineered based upon the anticipated loading and use. Post-tension slab underlayment and concrete mix for post-tension slabs and beams should conform the recommendations provided in the "Soil Moisture Considerations" section of this report. Specific soil presaturation is not required for very low expansive soils; however, the moisture content of the subgrade soils should be equal to or greater than the soils' optimum moisture content to a depth of 12 inches below grade prior to vapor retarder placement.

Post-Tensioning Institute (PTI) Method

Post-tensioned slabs should have sufficient stiffness to resist excessive bending due to non-uniform swell and shrinkage of subgrade soils. The differential movement can occur at the corner, edge, or center of slab. The potential for differential uplift can be evaluated based on design specifications of the PTI. The following table presents suggested minimum coefficients to be used in the PTI design method.

Thornthwaite Moisture Index	-20 inches/year
Correction Factor for Irrigation	20 inches/year
Depth to Constant Soil Suction	5 feet
Constant soil Suction (pf)	3.6
Moisture Velocity	0.75 inches/month

The coefficients are considered minimums and may not be adequate to represent worst case conditions such as adverse drainage and/or improper landscaping and maintenance. The above parameters are applicable provided structures have positive drainage that is maintained away from structures. Therefore, it is important that information regarding drainage, site maintenance, settlements, and effects of expansive soils be passed on to future owners.

Based on the above parameters, the following values were obtained from the Post-tensioning Institute 3rd Edition. The values may not be appropriate to account for possible differential settlement of the slab due to other factors. If a stiffer slab is desired, higher values of y_m may be warranted.

EXPANSION POTENTIAL	VERY LOW(3) TO LOW EXPANSIVE (E.I. = 0-50)
e_m center lift	5.0 feet
e_m edge lift	3.5 feet
y_m center lift	1.7 inches
y_m edge lift	0.75 inch
Bearing Value (1)	$\leq$ 3,000 psf
Lateral Pressure	250 psf
Subgrade Modulus (k)	100 pci/inch
Perimeter Footing Embedment (2)	12 inches

(1) Internal bearing values within the perimeter of the post-tension slab may be increased to 2,000 psf for a minimum embedment of 12 inches, then by 20 percent for each additional foot of embedment to a maximum of 3,000 psf.
(2) As measured below the lowest adjacent compacted subgrade surface.
(3) Foundations for very low expansive soil conditions may use the California Method (spanability method).
Note: The use of open bottomed raised planters adjacent to foundations will require more onerous design parameters.

Deepened footings/edges around the slab perimeter must be used to minimize non-uniform surface moisture migration (from an outside source) beneath the slab. An edge depth of 12 should be considered a minimum for very low expansive soils, respectively. The bottom of the deepened footing/edge should be designed to resist tension, using cable or reinforcement per the structural engineer. Other applicable recommendations presented under conventional foundation and the California Foundation Slab Method should be adhered to during the design and construction phase of the project.

PRELIMINARY FOUNDATION SETTLEMENT EVALUATION

Slabs and foundations should be designed to minimally accommodate an estimated 1 inch of differential settlement (angular distortion of 1/480), in 40 feet.

SOIL MOISTURE CONSIDERATIONS

GSI has evaluated the potential for vapor or water transmission through the slabs, in light of typical commercial floor coverings and improvements. Please note that typical slab moisture emission rates, range from about 2 to 27 lbs/24 hours/1,000 square feet from a 4-inch slab (Kanare, 2005), while most floor covering manufacturers recommend about 3 lbs/24 hours as an upper limit. Thus, the Client will need to evaluate the following in light of a cost v. benefit analysis (owner/occupant complaints and repairs/replacement), along with disclosure to all interested/affected parties.

Considering the E.I. test results of a very low expansion potential, anticipated typical water vapor transmission rates, floor coverings and improvements (to be chosen by the Client) that can tolerate those rates without distress, the following alternatives are provided.

- Concrete slab underlayment should consist of a 10-mil to 15-mil vapor retarder, or equivalent, with all laps sealed per the 2007 CBC (CBSC, 2007), ASTM E 1643, and the manufacturer's recommendation. The vapor retarder should comply with the ASTM E 1745 - Class A or B criteria, and be installed in accordance with ASTM E 1643.

- The 10- to 15-mil vapor retarder (ASTM E 1745 - Class A or B) shall be installed per the recommendations of the manufacturer and ASTM E 1643, including all penetrations (i.e., pipe, ducting, rebar, etc.).

- The vapor retarder may be placed directly on properly compacted subgrade soils with a very low expansion potential, and should be overlain by a 2-inch thick layer of clean sand (SE>30). If angular rocky soils are exposed at finish grade, the vapor retarder should be underlain by 2 inches of clean sand (SE>30) to provide a capillary break and/or to reduce puncture of the vapor retarder.

- Concrete should have a maximum water/cement ratio of 0.50. Additional concrete mix design recommendations should be provided by the structural consultant and/or waterproofing specialist. Concrete finishing and workability should be addressed by the structural consultant and a waterproofing specialist.

- Where slab water/cement ratios are as indicated above, and/or admixtures used, the structural consultant should also make changes to the concrete in the grade beams and footings in kind, so that the concrete used in the foundation and slabs are designed and/or treated for more uniform moisture protection.

- Building owner(s) should be specifically advised which areas are suitable for tile flooring, wood flooring, or other types of water/vapor-sensitive flooring and which are not suitable. In all planned floor areas, flooring shall be installed per the manufactures recommendations.

- Additional recommendations regarding water or vapor transmission should be provided by the architect/structural engineer/slab or foundation designer and should be consistent with the specified floor coverings indicated by the architect.

Regardless of the mitigation, some limited moisture/moisture vapor transmission through the slab should be anticipated. Construction crews may require special training for installation of certain product(s), as well as concrete finishing techniques. The use of specialized product(s) should be approved by the slab designer and water-proofing consultant. A technical representative of the flooring contractor should review the slab and moisture retarder plans and provide comment prior to the construction of the commercial foundations or improvements. The vapor retarder contractor should have representatives onsite during the initial installation.

WALL DESIGN PARAMETERS

General

The following section is for generalized geotechnical retaining wall recommendations on this site. Recommendations for specialty walls (i.e., grid-reinforced walls) or other specific cases, may be provided upon request. Walls below grade should be water-proofed, and should be free draining.

Conventional Retaining Walls

The design parameters provided below assume that either non expansive soils (typically Class 2 permeable filter material or Class 3 aggregate base) or native onsite materials (up to and including an E.I. of 50) are used to backfill any retaining walls. The type of backfill (i.e., select or native), should be specified by the wall designer, and clearly shown on the plans. The foundation system for the proposed retaining walls should be designed in accordance with the recommendations presented in this and preceding sections of this report, as appropriate. Footings should be embedded a minimum of 18 inches below adjacent grade (excluding landscape layer, 6 inches) and should be 24 inches in width. There should be no increase in bearing for footing width. Footing dimensions for building walls should consider the appropriate loading conditions previously indicated.

Restrained Walls/Loading Dock Walls

Any retaining walls that will be restrained prior to placing and compacting backfill material or that have re-entrant or male corners, should be designed for an at-rest equivalent fluid pressure (EFP) of 65 pcf, plus any applicable surcharge loading. For areas of male or re-entrant corners, the restrained wall design should extend a minimum distance of twice the height of the wall (2H) laterally from the corner. GSI recommends that loading dock walls be designed for a restrained condition. Temporary loads of stockpiled goods/supplies adjacent to the wall, should be applied to wall pressures. GSI recommends that 40 percent of these temporary loads should be added to the lateral pressures in order to account for these transient loading conditions.

Cantilevered Walls

The recommendations presented below are for cantilevered retaining walls up to 15 feet high. Design parameters for walls less than 3 feet in height may be superceded by County standard design. Active earth pressure may be used for retaining wall design, provided the top of the wall is not restrained from minor deflections. An equivalent fluid pressure approach may be used to compute the horizontal pressure against the wall. Appropriate fluid unit weights are given below for specific slope gradients of the retained material. These do not include other superimposed loading conditions due to traffic, structures, seismic events or adverse geologic conditions. When wall configurations are finalized, the appropriate loading conditions for superimposed loads can be provided upon request.

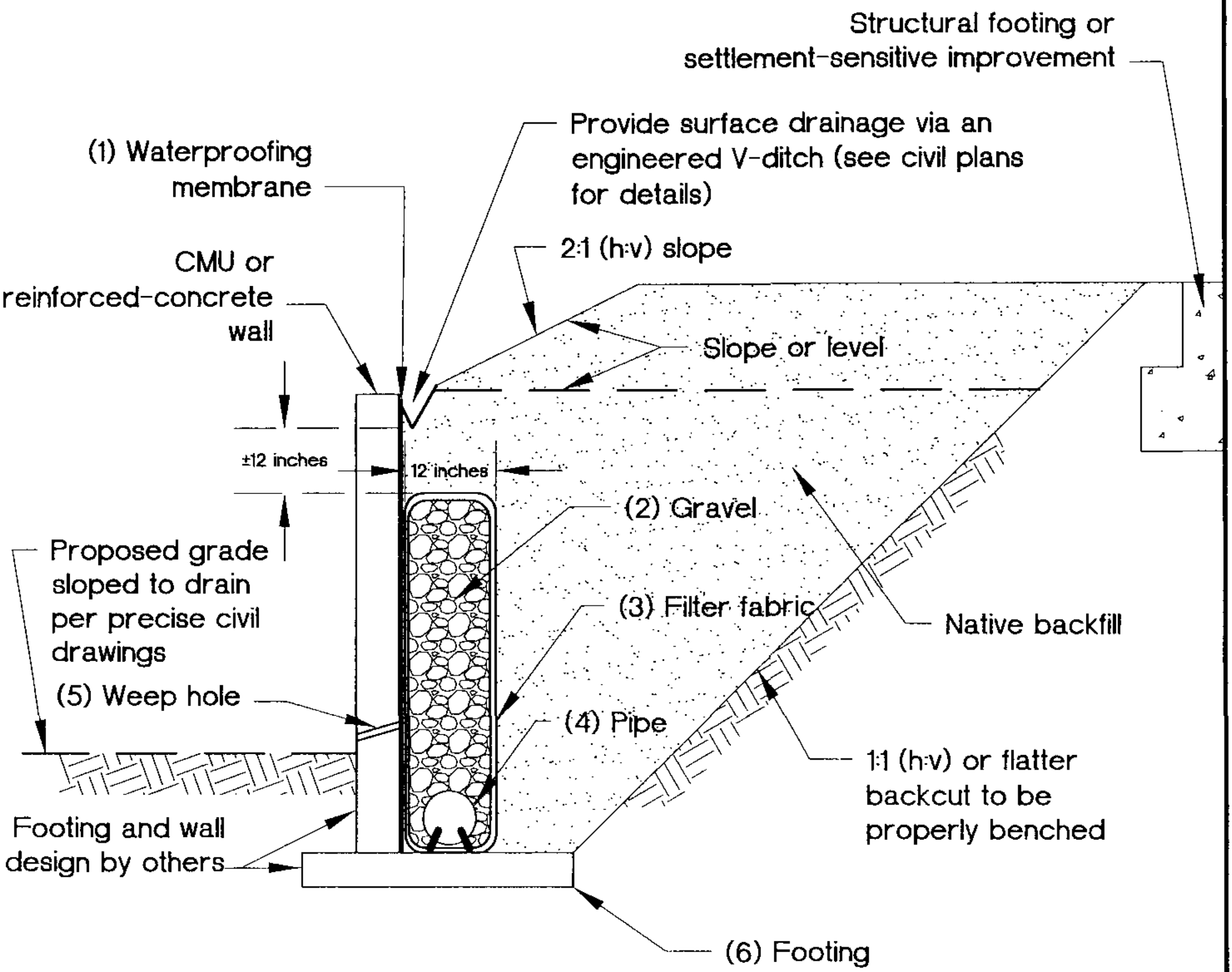
For truck dock walls and site retaining walls exceeding 6 feet in height, the proposed retaining walls should be minimally designed with a seismic surcharge of 10H, where "H" is the height of the retained soil from the bottom of the footing (excluding shear key), to the top of the backfill. This should be applied as a uniform pressure to the active wall loads and achieve a seismic overturning factor-of-safety of 1.2. For the evaluation of seismic surcharge, the bearing pressure may exceed the static value by one-third considering the transient nature of this surcharge.

SURFACE SLOPE OF RETAINED MATERIAL (HORIZONTAL:VERTICAL)	EQUIVALENT FLUID WEIGHT P.C.F. (SELECT [PRE-APPROVED] BACKFILL**)	EQUIVALENT FLUID WEIGHT P.C.F. (NATIVE BACKFILL)***
Level*	38	48
2 to 1	55	65

* Level backfill behind a retaining wall is defined as compacted earth materials, properly drained, without a slope for a distance of 2H behind the wall, where H is the height of the wall.
** SE $\geq$35, P.I. <15, E.I. <21, -200 ($\leq$10%).
*** E.I. between 0 - 50, P.I. <15, - 200 (<15%).

Retaining Wall Backfill and Drainage

Positive drainage must be provided behind all retaining walls in the form of gravel wrapped in geofabric and outlets. A backdrain system is considered necessary for retaining walls that are 2 feet or greater in height. Details 1, 2, and 3, present the back drainage options discussed below. Backdrains should consist of a 4-inch diameter perforated PVC or ABS pipe encased in either Class 2 permeable filter material or ¾-inch to 1½-inch gravel wrapped in approved filter fabric (Mirafi 140, or equivalent). Backdrains should be constructed to drain toward a suitable outlet via gravity. If gravity flow toward a suitable outlet cannot be attained, sump pumps may be necessary. Sump pumps should be



(1) Waterproofing membrane.

(2) Gravel: Clean, crushed, ¾ to 1½ inch.

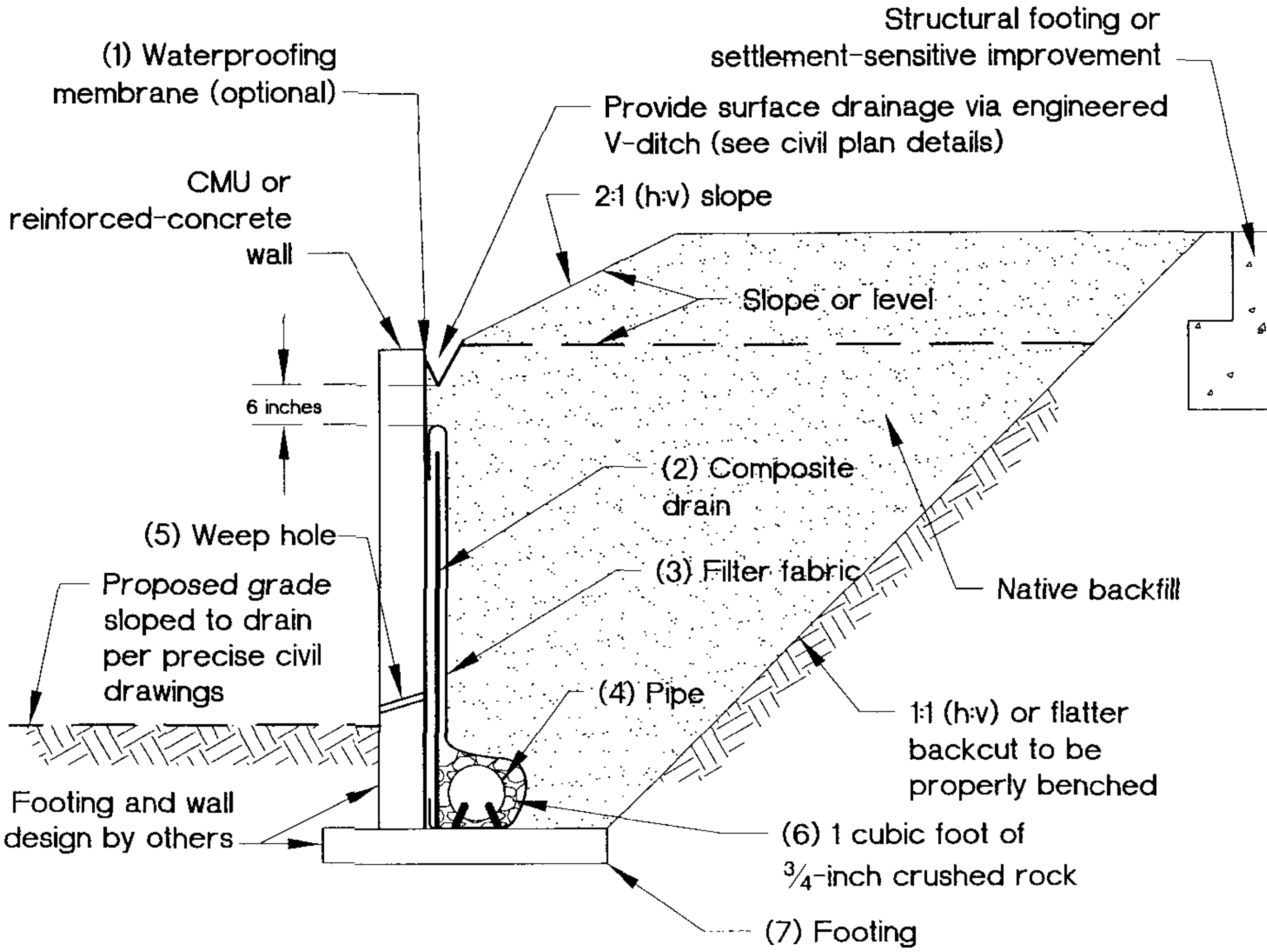
(3) Filter fabric: Mirafi 140N or approved equivalent.

(4) Pipe: 4-inch-diameter perforated PVC, Schedule 40, or approved alternative with minimum of 1 percent gradient sloped to suitable, approved outlet point (perforations down).

(5) Weep hole: Minimum 2-inch diameter placed at 20-foot centers along the wall and placed 3 inches above finished surface. Design civil engineer to provide drainage at toe of wall. No weep holes for below-grade walls.

(6) Footing: If bench is created behind the footing greater than the footing width, use level fill or cut natural earth materials. An additional "heel" drain will likely be required by geotechnical consultant.

GeoSoils, Inc.	RETAINING WALL DETAIL – ALTERNATIVE A	Detail 1



(1) Waterproofing membrane (optional): Liquid boot or approved mastic equivalent.

(2) Drain: Miradrain 6000 or J-drain 200 or equivalent for non-waterproofed walls; Miradrain 6200 or J-drain 200 or equivalent for waterproofed walls (all perforations down).

(3) Filter fabric: Mirafi 140N or approved equivalent; place fabric flap behind core.

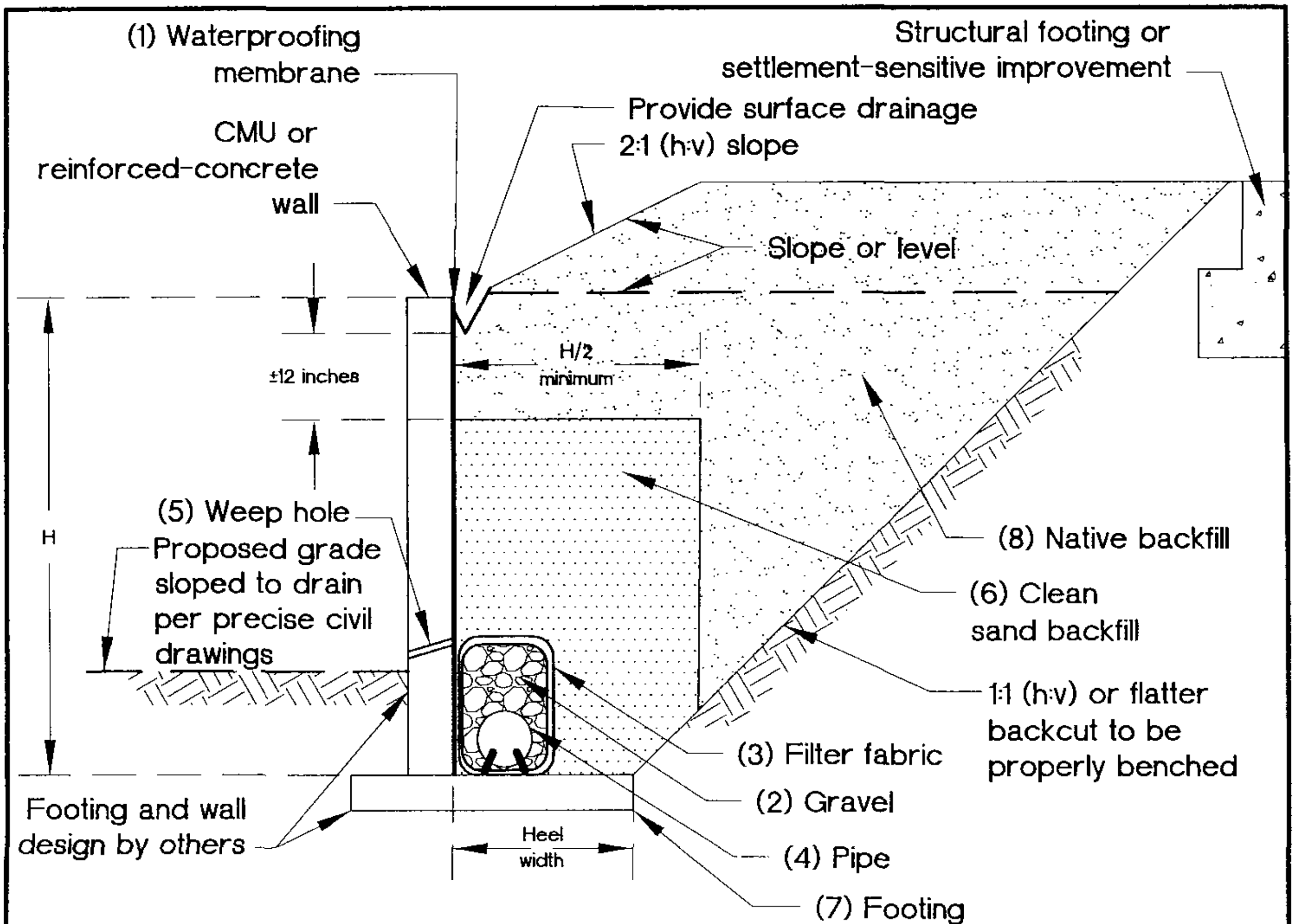
(4) Pipe: 4-inch-diameter perforated PVC, Schedule 40, or approved alternative with minimum of 1 percent gradient to proper outlet point (perforations down).

(5) Weep hole: Minimum 2-inch diameter placed at 20-foot centers along the wall and placed 3 inches above finished surface. Design civil engineer to provide drainage at toe of wall. No weep holes for below-grade walls.

(6) Gravel: Clean, crushed, ¾ to 1½ inch.

(7) Footing: If bench is created behind the footing greater than the footing width, use level fill or cut natural earth materials. An additional "heel" drain will likely be required by geotechnical consultant.

GeoSoils, Inc.	RETAINING WALL DETAIL – ALTERNATIVE B	Detail 2



(1) Waterproofing membrane: Liquid boot or approved masticequivalent.

(2) Gravel: Clean, crushed, $\frac{3}{4}$ to $1\frac{1}{2}$ inch.

(3) Filter fabric: Mirafi 140N or approved equivalent.

(4) Pipe: 4-inch-diameter perforated PVC, Schedule 40, or approved alternative with minimum of 1 percent gradient to proper outlet point (perforations down).

(5) Weep hole: Minimum 2-inch diameter placed at 20-foot centers along the wall and placed 3 inches above finished surface. Design civil engineer to provide drainage at toe of wall. No weep holes for below-grade walls.

(6) Clean sand backfill: Must have sand equivalent value (S.E.) of 35 or greater; can be densified by water jetting upon approval by geotechnical engineer.

(7) Footing: If bench is created behind the footing greater than the footing width, use level fill or cut natural earth materials. An additional "heel" drain will likely be required by geotechnical consultant.

(8) Native backfill: If E.I. <21 and S.E. ≥ 35 then all sand requirements also may not be required and will be reviewed by the geotechnical consultant.

GeoSoils, Inc.	RETAINING WALL DETAIL — ALTERNATIVE C	Detail 3

designed by the project architect to prevent the local saturation of the surrounding soils. For low expansive backfill, the filter material should extend a minimum of 1 horizontal foot behind the base of the walls and upward at least 1 foot. For native backfill that has up to medium expansion potential, continuous Class 2 permeable drain materials should be used behind the wall. This material should be continuous (i.e., full height) behind the wall, and it should be constructed in accordance with the enclosed Detail 1 (Typical Retaining Wall Backfill and Drainage Detail). For limited access and confined areas, (panel) drainage behind the wall may be constructed in accordance with Detail 2 (Retaining Wall Backfill and Subdrain Detail Geotextile Drain). All below-grade walls should be waterproofed. Materials with an E.I. potential of greater than 50 should not be used as backfill for retaining walls. For more onerous expansive situations, backfill and drainage behind the retaining wall should conform with Detail 3 (Retaining Wall And Subdrain Detail Clean Sand Backfill).

Outlets should consist of a 4-inch diameter solid PVC or ABS pipe spaced no greater than $\pm$100 feet apart, with a minimum of two outlets, one on each end. The use of weep holes, only, in walls higher than 2 feet, is not recommended. The surface of the backfill should be sealed by pavement or the top 18 inches compacted with native soil (E.I. $\leq$90). Proper surface drainage should also be provided. For additional mitigation, consideration should be given to applying a water-proof membrane to the back of all retaining structures. The use of a waterstop should be considered for all concrete and masonry joints.

Wall/Retaining Wall Footing Transitions

Site walls are anticipated to be founded on footings designed in accordance with the recommendations in this report. Should wall footings transition from cut to fill, the civil designer may specify either:

a) A minimum of a 2-foot overexcavation and recompaction of cut materials for a distance of 2H, from the point of transition.

b) Increase of the amount of reinforcing steel and wall detailing (i.e., expansion joints or crack control joints) such that a angular distortion of 1/360 for a distance of 2H on either side of the transition may be accommodated. Expansion joints should be placed no greater than 20 feet on-center, in accordance with the structural engineer's/wall designer's recommendations, regardless of whether or not transition conditions exist. Expansion joints should be sealed with a flexible, non-shrink grout.

c) Embed the footings entirely into native bedrock material (i.e., deepened footings).

If transitions from cut to fill transect the wall footing alignment at an angle of less than 45 degrees (plan view), then the designer should follow recommendation "a" (above) and until such transition is between 45 and 90 degrees to the wall alignment.

TOP-OF-SLOPE WALLS/FENCES/IMPROVEMENTS

Slope Creep

Some of the soils at the site may be expansive and therefore, may become desiccated when allowed to dry. Such soils are susceptible to surficial slope creep, especially with seasonal changes in moisture content. Typically in southern California, during the hot and dry summer period, these soils become desiccated and shrink, thereby developing surface cracks. The extent and depth of these shrinkage cracks depend on many factors such as the nature and expansivity of the soils, temperature and humidity, and extraction of moisture from surface soils by plants and roots. When seasonal rains occur, water percolates into the cracks and fissures, causing slope surfaces to expand, with a corresponding loss in soil density and shear strength near the slope surface. With the passage of time and several moisture cycles, the outer 3 to 5 feet of slope materials experience a very slow, but progressive, outward and downward movement, known as slope creep. For slope heights greater than 10 feet, this creep related soil movement will typically impact all rear yard flatwork and other secondary improvements that are located within about 15 feet from the top of slopes, such as concrete flatwork, etc., and in particular top of slope fences/walls. This influence is normally in the form of detrimental settlement, and tilting of the proposed improvements. The dessication/swelling and creep discussed above continues over the life of the improvements, and generally becomes progressively worse. Accordingly, the developer should provide this information to any all interested/affected parties.

Top of Slope Walls/Fences

Due to the potential for slope creep for slopes higher than about 10 feet, some settlement and tilting of the walls/fence with the corresponding distresses, should be expected. To mitigate the tilting of top of slope walls/fences, we recommend that the walls/fences be constructed on deepened foundations without any consideration for creep forces, where the expansion index of the materials comprising the outer 15 feet of the slope is less than 50. The strength of the concrete and grout should be evaluated by the structural engineer of record. The proper ASTM tests for the concrete and mortar should be provided along with the slump quantities. The concrete used should be appropriate to mitigate sulfate corrosion, as warranted.

DRIVEWAY APRONS, FLATWORK, AND OTHER IMPROVEMENTS

Some of the site soil materials on site may be expansive. The effects of expansive soils are cumulative, and typically occur over the lifetime of any improvements. On relatively level areas, when the soils are allowed to dry, the dessication and swelling process tends to cause heaving and distress to flatwork and other improvements. The resulting potential for distress to improvements may be reduced, but not totally eliminated. To that end, it is recommended that the developer should notify all interested/affected parties of this long-term potential for distress. To reduce the likelihood of distress, the following recommendations are presented for all exterior flatwork:

1. The subgrade area for concrete slabs should be compacted to achieve a minimum 90 percent relative compaction, and then be presoaked to 2 to 3 percentage points above (or 125 percent of) the soils' optimum moisture content, to a depth of 18 inches below subgrade elevation. If very low expansive soils are present, only optimum moisture content, or greater, is required and specific presoaking is not warranted. The moisture content of the subgrade should be proof tested within 72 hours prior to pouring concrete.

2. Concrete slabs should be cast over a non-yielding surface, consisting of a 4-inch layer of crushed rock, gravel, or clean sand, that should be compacted and level prior to pouring concrete. The layer or subgrade should be wet-down completely prior to pouring concrete, to minimize loss of concrete moisture to the surrounding earth materials.

3. Exterior slabs should be a minimum of 4 inches thick. Driveway slabs and approaches should additionally have a thickened edge (12 inches) adjacent to all landscape areas, to help impede infiltration of landscape water under the slab.

4. The use of transverse and longitudinal control joints are recommended to help control slab cracking due to concrete shrinkage or expansion. Two ways to mitigate such cracking are: a) add a sufficient amount of reinforcing steel, increasing tensile strength of the slab; and, b) provide an adequate amount of control and/or expansion joints to accommodate anticipated concrete shrinkage and expansion.

   In order to reduce the potential for unsightly cracks, slabs should be reinforced at mid-height with a minimum of No. 3 bars placed at 18 inches on center, in each direction. If subgrade soils within the top 7 feet from finish grade are very low expansive soils (i.e., E.I. $\leq$20), then 6x6-W1.4xW1.4 welded-wire mesh may be substituted for the rebar, provided the reinforcement is placed on chairs, at slab mid-height. The exterior slabs should be scored or saw cut, ½ to ⅜ inches deep, often enough so that no section is greater than 10 feet by 10 feet. For sidewalks or narrow slabs, control joints should be provided at intervals of every 6 feet. The slabs should be separated from the foundations and sidewalks with expansion joint filler material.

5. No traffic should be allowed upon the newly poured concrete slabs until they have been properly cured to within 75 percent of design strength. Concrete compression strength should be a minimum of 2,500 psi.

6. Sidewalks, and apron slabs adjacent to the building should be separated from the building with thick expansion joint filler material. In areas directly adjacent to a continuous source of moisture (i.e., irrigation, planters, etc.), all joints should be additionally sealed with flexible mastic.

7. Planters and walls should not be tied to the structure.

8. Overhang structures should be supported on the slabs, or structurally designed with continuous footings tied in at least two directions. If very low expansion soils are present, footings need only be tied in one direction.

9. Any masonry landscape walls that are to be constructed throughout the property should be grouted and articulated in segments no more than 20 feet long. These segments should be keyed or doweled together.

10. Utilities should be enclosed within a closed utilidor (vault) or designed with flexible connections to accommodate differential settlement and/or expansive soil conditions.

11. Positive site drainage should be maintained at all times. Finish grade on the lots should provide a minimum of 1 to 2 percent fall to the street, as indicated herein, or per code, whichever is more onerous. It should be kept in mind that drainage reversals could occur, including post-construction settlement, if relatively flat landscape area drainage gradients are not periodically maintained.

12. If air conditioning (A/C) units are not placed on the roof(s) of the structure(s), they should be supported by slabs that are incorporated into the building foundation or constructed on a rigid slab with flexible couplings for plumbing and electrical lines. A/C waste water lines should be drained to a suitable non-erosive outlet.

13. Shrinkage cracks could become excessive if proper finishing and curing practices are not followed. Finishing and curing practices should be performed per the Portland Cement Association Guidelines. Mix design should incorporate rate of curing for climate and time of year, sulfate content of soils, corrosion potential of soils, and fertilizers used on site.

PRELIMINARY PAVEMENT DESIGN

For preliminary pavement design, GSI has incorporated the resistance value (R-value) determined from laboratory testing and assumed Traffic Indices (TI). Pavement sections consisting of asphaltic concrete (AC) over aggregate base and full depth Portland cement concrete (PCC) pavement are provided. The following preliminary recommendations are for private right-of-ways only. Street work within the public right-of-way may require higher standards. Final pavement design should be re-evaluated based upon actual R-value testing after the completion of grading and underground utility trench backfill. This preliminary AC pavement design analysis can be revised if the actual T.I. differs from the assumed values.

AC PAVEMENTS				
LOCATION	**SUBGRADE (R-VALUE)**	**TRAFFIC INDEX (T.I.)**	**ASPHALTIC CONCRETE THICKNESS (INCHES)**	**AGGREGATE BASE (1) THICKNESS (INCHES)**
Parking Stalls	77	5.0	4.0	6.0 (2)
Drive Areas/Fire Lane/Loading Docks	77	6.0	4.0	6.0 (2)

(1) - Denotes Class 2 Aggregate Base (R≥78, SE ≥22)
(2) - GSI does not recommend less than 6 inches of aggregate base for any pavement application

PCC PAVEMENTS				
TRAFFIC AREA	**AVERAGE DAILY(2) TRUCK TRAFFIC (ADTT Assumed)**	**SUBGRADE R-VALUE(3)**	**AXLE LOAD CATEGORY**	**P.C.C.(1) THICKNESS (Inches) 560-C-3250**
Gas Station Pump Island	25	50 to 77	Light Moderate Heavy	6.0 6.5 7.0

(1) Concrete shall be 560-C-3250. Assumes the construction of concrete shoulders and/or a transition to AC pavement.
(2) ADTT values have been assumed for planning purposes herein and should be confirmed by the design team during future plan development.
(3) R-values should be verified upon completion of site grading.

The PCC pavement sections presented above have been calculated based on an anticipated subgrade R-value of 50 to 77, modulus of rupture (MR) of 500 psi, and the use of concrete shoulders (curb and gutter) or AC pavement at the edge of the concrete pavement. Load safety factors of 1.0, 1.1, and1.2 was applied in the analysis. At this time, GSI anticipates an ADTT value of 25 or less for the proposed concrete pavement section. This analysis should be revised when actual ADTT values have been provided by the project traffic engineer.

The transition of PCC to AC pavements should be provided with an additional 6-inch thick by 5-foot wide thickened edge of PCC or a 5-foot wide zone of full depth AC to provide a cut-off wall to transfer wheel loads and reduce the potential for pavement subgrade pumping (below the pavement sections). A minimum 4-inch layer of base rock in loading dock areas should be considered to improve pavement performance. Base rock may consist of either ¾-inch crushed rock or Caltrans Class 2 aggregate base. Crushed rock may be compacted by vibratory methods. Aggregate base should be compacted to a minimum relative compaction of 95 percent.

All pavement installation, including preparation and compaction of subgrade, compaction of base material, and placement and rolling of asphaltic concrete, should be done in accordance with County of San Diego guidelines and under the observation and testing services provided by the project geotechnical engineer and/or the County of San Diego.

The recommended pavement section provided above is intended as a minimum guideline. If thinner or highly variable pavement sections are constructed, increased maintenance and repair could be expected. If the ADT (average daily traffic) or ADTT (average daily truck traffic) increases beyond that intended, as reflected by the T.I. used for design, increased maintenance and repair could be required for the pavement section. Consideration should be given to the increased potential for distress from overuse of paved street areas by heavy equipment and/or construction related heavy traffic (e.g., concrete trucks, loaded supply trucks, etc.), particularly when the final section is not in place (i.e., topcoat). Best management construction practices should be followed at all times, especially during inclement weather.

AC PAVEMENT GRADING RECOMMENDATIONS

General

All section changes should be properly transitioned. If adverse conditions are encountered during the preparation of subgrade materials, special construction methods may need to be employed. A GSI representative should be present for the preparation of subgrade, base rock, and asphalt concrete.

Subgrade

Within street, and onsite drive and parking areas, all surficial deposits of loose soil material should be removed and recompacted as recommended. After the loose soils are removed, the bottom is to be scarified to a depth of at least 6 inches, moisture conditioned as necessary and compacted to 95 percent of the maximum laboratory density (ASTM D 1557) or the County of San Diego minimum, as determined by ASTM D 1557. The subgrade soils beneath curb and gutters, cross-gutters, and driveway approaches should also be compacted to 95 percent of the laboratory standard (ASTM D 1557).

Deleterious material, excessively wet or dry pockets, concentrated zones of oversized rock fragments, and any other unsuitable materials encountered during grading should be removed. The compacted fill material should then be brought to the elevation of the proposed subgrade for the pavement. The subgrade should be proof-rolled in order to ensure a uniform firm and unyielding surface. All grading and fill placement should be observed by the project soil engineer and/or his representative.

Aggregate Base Rock

Compaction tests are required for the recommended base section. Minimum relative compaction required will be 95 percent of the laboratory maximum density as determined by ASTM test method D-1557 and/or Caltrans Test Method Number California 216. Base aggregate should be in accordance with the Caltrans Class 2 base rock (minimum R-value=78). A minimum of 6 inches of aggregate base compacted to 95 percent of the laboratory standard (ASTM D-1557) should be provided beneath all concrete curb and gutters, concrete cross-gutters, and concrete, driveway approaches.

Paving

Prime coat may be omitted if all of the following conditions are met:

1. The asphalt pavement layer is placed within two weeks of completion of base and/or subbase course.

2. Traffic is not routed over completed base before paving.

3. Construction is completed during the dry season of May through October.

4. The base is kept free of debris prior to placement of asphaltic concrete.

If construction is performed during the wet season of November through April, prime coat may be omitted if no rain occurs between completion of base course and paving <u>and</u> the time between completion of base and paving is reduced to three days, provided the base is free of loose soil or debris. Where prime coat has been omitted and rain occurs, traffic is routed over base course, or paving is delayed, measures shall be taken to restore base course, and subgrade to conditions that will meet specifications as directed by the soil engineer.

Drainage

Positive drainage should be provided for all surface water to drain towards the area swale, curb and gutter, or to an approved drainage channel. Positive site drainage should be maintained at all times. Water should not be allowed to pond or seep into the ground. If planters or landscaping are adjacent to paved areas, measures should be taken to minimize the potential for water to enter the pavement section, such as enclosed planters, thickened edges, etc.

PCC Pavement Joints

Weakened Plane Joints

Transverse and longitudinal weakened plane joints may be constructed per Caltrans Standard specifications, Section 40-1.08B and 40-1.08B(1). Transverse weakened plane joints should be spaced no farther than 15 feet apart and no closer than 5 feet. Longitudinal weakened plane joints should be spaced no farther than 20 feet apart, but not less than 5 feet. Joint layout may be determined per the applicable San Diego Regional Standard Drawings G-18 through G-21 (inclusive).

Expansion Joints

Transverse expansion joints should be constructed at 120-foot spacings in accordance with San Diego Regional Standard Drawing G-10.

Contact Joints

Transverse and longitudinal contact joints should be constructed in accordance with the contact joint detail shown on the San Diego Regional Standard Drawing G-10. Joint layout may be determined per the applicable San Diego Regional Standard Drawings G-18 through G-21 (inclusive). Within large parking areas, joint spacings should be no greater than 20 feet.

Slab Reinforcement

PCC Pavements for this project are designed as unreinforced and should perform adequately, assuming proper construction. If additional control of internal slab stresses (i.e., curing shrinkage, thermal expansion and contraction), and the effects of expansive soil subgrades is desired, then the use of No. 4 reinforcing bars, 12 inches on center each way, should be considered.

Subgrade should be compacted to a minimum relative compaction of 90 percent. Aggregate base compaction should be 95 percent of the maximum dry density (ASTM D 1557). If adverse conditions (i.e., saturated ground, etc.) are encountered during preparation of subgrade, special construction methods may need to be employed. These recommendations should be considered preliminary. R-value testing and pavement design analysis should be performed upon completion of grading for the pad.

DEVELOPMENT CRITERIA

Slope Deformation

Compacted fill slopes designed using customary factors of safety for gross or surficial stability and constructed in general accordance with the design specifications should be expected to undergo some differential vertical heave or settlement in combination with differential lateral movement in the out-of-slope direction, after grading. This post-construction movement occurs in two forms: slope creep, and lateral fill extension (LFE). Slope creep is caused by alternate wetting and drying of the fill soils which results in slow downslope movement. This type of movement is expected to occur throughout the life of the slope, and is anticipated to potentially affect improvements or structures (e.g., separations and/or cracking), placed near the top-of-slope, up to a maximum distance of approximately 15 feet from the top-of-slope, depending on the slope height. This movement generally results in rotation and differential settlement of improvements located within the creep zone. LFE occurs due to deep wetting from irrigation and rainfall on slopes comprised of expansive materials. Although some movement should be expected, long-term movement from this source may be minimized, but not eliminated, by placing the fill throughout the slope region, wet of the fill's optimum moisture content.

It is generally not practical to attempt to eliminate the effects of either slope creep or LFE. Suitable mitigative measures to reduce the potential of lateral deformation typically include: setback of improvements from the slope faces (per the 1997 UBC and/or adopted California Building Code), positive structural separations (i.e., joints) between improvements, and stiffening and deepening of foundations. Expansion joints in walls should be placed no greater than 20 feet on-center, and in accordance with the structural engineer's recommendations. All of these measures are recommended for design of structures and improvements. The ramifications of the above conditions, and recommendations for mitigation, should be provided to all interested/affected parties.

Slope Maintenance and Planting

Water has been shown to weaken the inherent strength of all earth materials. Slope stability is significantly reduced by overly wet conditions. Positive surface drainage away from slopes should be maintained and only the amount of irrigation necessary to sustain plant life should be provided for planted slopes. Over-watering should be avoided as it adversely affects site improvements, and causes perched groundwater conditions. Graded slopes constructed utilizing onsite materials would be erosive. Eroded debris may be minimized and surficial slope stability enhanced by establishing and maintaining a suitable vegetation cover soon after construction. Compaction to the face of fill slopes would tend to minimize short-term erosion until vegetation is established. Plants selected for landscaping should be light weight, deep rooted types that require little water and are capable of surviving the prevailing climate. Jute-type matting or other fibrous covers may aid in allowing the establishment of a sparse plant cover. Utilizing plants other than those recommended above will increase the potential for perched water, staining, mold, etc., to develop. A rodent control program to prevent burrowing should be implemented. Irrigation of natural (ungraded) slope areas is generally not recommended. These recommendations regarding plant type, irrigation practices, and rodent control should be provided to the property owner. Over-steepening of slopes should be avoided during building construction activities and landscaping.

Drainage

Adequate lot surface drainage is a very important factor in reducing the likelihood of adverse performance of foundations, hardscape, and slopes. Surface drainage should be sufficient to prevent ponding of water anywhere on a lot, and especially near structures and tops of slopes. Lot surface drainage should be carefully taken into consideration during fine grading, landscaping, and building construction. Therefore, care should be taken that future landscaping or construction activities do not create adverse drainage conditions. Positive site drainage within lots and common areas should be provided and maintained at all times. Drainage should not flow uncontrolled down any descending slope. Water should be directed away from foundations and not allowed to pond and/or seep into the ground. In general, the area within 5 feet around a structure should slope away from the structure. We recommend that unpaved lawn and landscape areas have a minimum gradient of 1 percent sloping away from structures, and whenever possible, should be above adjacent paved areas, or comply with code, whichever is more onerous. Consideration should be given to avoiding construction of planters adjacent to structures (i.e., buildings, etc.). Pad drainage should be directed toward the street or other approved area(s). Although not a geotechnical requirement, roof gutters, down spouts, or other appropriate means may be utilized to control roof drainage. Down spouts, or drainage devices should outlet a minimum of 5 feet from structures or into a subsurface drainage system. Areas of seepage may develop due to irrigation or heavy rainfall, and should be anticipated. Minimizing irrigation will lessen this potential. If areas of seepage develop, recommendations for minimizing this effect could be provided upon request.

Erosion Control

Graded slopes will be subject to surficial erosion during and after grading. Onsite earth materials have a high erosion potential. Consideration should be given to providing hay bales and silt fences for the temporary control of surface water, from a geotechnical viewpoint.

Landscape Maintenance

Only the amount of irrigation necessary to sustain plant life should be provided. Over-watering the landscape areas will adversely affect proposed site improvements. We would recommend that any proposed open-bottom planters adjacent to proposed structures be eliminated for a minimum distance of 10 feet. As an alternative, closed-bottom type planters could be utilized. An outlet placed in the bottom of the planter, could be installed to direct drainage away from structures or any exterior concrete flatwork. If planters are constructed adjacent to structures, the sides and bottom of the planter should be provided with a moisture retarder to prevent penetration of irrigation water into the subgrade. Provisions should be made to drain the excess irrigation water from the planters without saturating the subgrade below or adjacent to the planters. Graded slope areas should be planted with drought resistant vegetation. Consideration should be given to the type of vegetation chosen and their potential effect upon surface improvements (i.e., some trees will have an effect on concrete flatwork with their extensive root systems). From a geotechnical standpoint leaching is not recommended for establishing landscaping. If the surface soils are processed for the purpose of adding amendments, they should be recompacted to 90 percent minimum relative compaction.

Gutters and Downspouts

As previously discussed in the drainage section, the installation of gutters and downspouts should be considered to collect roof water that may otherwise infiltrate the soils adjacent to the structures. If utilized, the downspouts should be drained into PVC collector pipes or other non-erosive devices (e.g., paved swales or ditches; below grade, solid tight-lined PVC pipes; etc.), that will carry the water away from the structure, to an appropriate outlet, in accordance with the recommendations of the design civil engineer. Downspouts and gutters are not a requirement; however, from a geotechnical viewpoint, provided that positive drainage is incorporated into project design (as discussed previously).

Subsurface and Surface Water

Subsurface and surface water are not anticipated to affect site development, provided that the recommendations contained in this report are incorporated into final design and construction and that prudent surface and subsurface drainage practices are incorporated into the construction plans. Perched groundwater conditions along zones of contrasting permeabilities may not be precluded from occurring in the future due to site irrigation, poor drainage conditions, or damaged utilities, and should be anticipated. Should perched groundwater conditions develop, this office could assess the affected area(s) and provide the appropriate recommendations to mitigate the observed groundwater conditions. Groundwater conditions may change with the introduction of irrigation, rainfall, or other factors.

Site Improvements

If in the future, any additional improvements are planned for the site, recommendations concerning the geological or geotechnical aspects of design and construction of said improvements could be provided upon request. Water features should not be constructed without specific design and construction recommendations from GSI, and this construction recommendation should be provided to all interested parties. This office should be notified in advance of any fill placement, grading of the site, or trench backfilling after rough grading has been completed. This includes any grading, utility trench and retaining wall backfills, flatwork, etc.

Tile Flooring

Tile flooring can crack, reflecting cracks in the concrete slab below the tile, although small cracks in a conventional slab may not be significant. Therefore, the designer should consider additional steel reinforcement for concrete slabs-on-grade where tile will be placed. The tile installer should consider installation methods that reduce possible cracking of the tile such as slipsheets. Slipsheets or a vinyl crack isolation membrane (approved by the Tile Council of America/Ceramic Tile Institute) are recommended between tile and concrete slabs on grade.

Additional Grading

This office should be notified in advance of any fill placement, supplemental regrading of the site, or trench backfilling after rough grading has been completed. This includes completion of grading in the street, driveway approaches, driveways, parking areas, and utility trench and retaining wall backfills.

Footing Trench Excavation

All footing excavations should be observed by a representative of this firm subsequent to trenching and prior to concrete form and reinforcement placement. The purpose of the observations is to evaluate that the excavations have been made into the recommended bearing material and to the minimum widths and depths recommended for construction. If loose or compressible materials are exposed within the footing excavation, a deeper footing or removal and recompaction of the subgrade materials would be recommended at that time. Footing trench spoil and any excess soils generated from utility trench excavations should be compacted to a minimum relative compaction of 90 percent, if not removed from the site.

Trenching/Temporary Construction Backcuts

Considering the nature of the onsite earth materials, it should be anticipated that caving or sloughing could be a factor in subsurface excavations and trenching. Shoring or excavating the trench walls/backcuts at the angle of repose (typically 25 to 45 degrees [except as specifically superceded within the text of this report]), should be anticipated. All excavations should be observed by an engineering geologist or soil engineer from GSI, prior to workers entering the excavation or trench, and minimally conform to Cal-OSHA, state, and local safety codes. Should adverse conditions exist, appropriate recommendations would be offered at that time. The above recommendations should be provided to any contractors and/or subcontractors, or property owners, etc., that may perform such work.

Utility Trench Backfill

1. All interior utility trench backfill should be brought to at least 2 percent above optimum moisture content and then compacted to obtain a minimum relative compaction of 90 percent of the laboratory standard. As an alternative for shallow (12-inch to 18-inch) under-slab trenches, sand having a sand equivalent value of 30 or greater may be utilized and jetted or flooded into place. Observation, probing and testing should be provided to evaluate the desired results.

2. Exterior trenches adjacent to, and within areas extending below a 1:1 plane projected from the outside bottom edge of the footing, and all trenches beneath hardscape features and in slopes, should be compacted to at least 90 percent of the laboratory standard. Sand backfill, unless excavated from the trench, should not be used in these backfill areas. Compaction testing and observations, along with probing, should be accomplished to evaluate the desired results.

3. All trench excavations should conform to Cal-OSHA, state, and local safety codes.

4. Utilities crossing grade beams, perimeter beams, or footings should either pass below the footing or grade beam utilizing a hardened collar or foam spacer, or pass through the footing or grade beam in accordance with the recommendations of the structural engineer.

Monitoring

Due to the proximity of the existing commercial developments on three sides of the site, GSI recommends that a pre-construction elevation survey and photographic documentation of the existing improvements be performed prior to development activities onsite. Based on the level, quality and age of the adjoining improvements, construction activities may induce vibrations, settlement or other impacts. Therefore, construction monitoring may be warranted and should be evaluated once the site layout is provided to this office.

SUMMARY OF RECOMMENDATIONS REGARDING GEOTECHNICAL OBSERVATION AND TESTING

We recommend that observation and/or testing be performed by GSI at each of the following construction stages:

- During grading/recertification.

- During excavation.

- During placement of subdrains, toe drains, or other subdrainage devices, prior to placing fill and/or backfill.

- After excavation of building footings, retaining wall footings, and free standing walls footings, prior to the placement of reinforcing steel or concrete.

- Prior to pouring any slabs or flatwork, after presoaking/presaturation of building pads and other flatwork subgrade, before the placement of concrete, reinforcing steel, capillary break (i.e., sand, pea-gravel, etc.), or vapor retarders (i.e., visqueen, etc.).

- During retaining wall subdrain installation, prior to backfill placement.

- During placement of backfill for area drain, interior plumbing, utility line trenches, and retaining wall backfill.

- During slope construction/repair.

- When any unusual soil conditions are encountered during any construction operations, subsequent to the issuance of this report.

- When any developer or property-owner improvements, such as flatwork, walls, etc., are constructed, prior to construction.

- A report of geotechnical observation and testing should be provided at the conclusion of each of the above stages, in order to provide concise and clear documentation of site work, and/or to comply with code requirements.

- GSI should review project sales documents for geotechnical aspects, including irrigation practices, the conditions outlined above, etc., prior to any sales. At that stage, GSI will provide property-owner maintenance guidelines which should be incorporated into such documents.

OTHER DESIGN PROFESSIONALS/CONSULTANTS

The design civil engineer, structural engineer, post-tension designer, architect, landscape architect, wall designer, etc., should review the recommendations provided herein, incorporate those recommendations into all their respective plans, and by explicit reference, make this report part of their project plans. This report presents minimum design criteria for the design of slabs, foundations and other elements possibly applicable to the project. These criteria should not be considered as substitutes for actual designs by the structural engineer/designer. Please note that the recommendations contained herein are not intended to preclude the transmission of water or vapor through the slab or foundation. The structural engineer/foundation and/or slab designer should provide recommendations to not allow water or vapor to enter into the structure so as to cause damage to another building component, or so as to limit the installation of the type of flooring materials typically used for the particular application. Property owners should be advised of the potential for water or water vapor transmission through foundations and slabs.

The structural engineer/designer should analyze actual soil-structure interaction and consider, as needed, bearing, expansive soil influence, and strength, stiffness and deflections in the various slab, foundation, and other elements in order to develop appropriate, design-specific details. As conditions dictate, it is possible that other influences will also have to be considered. The structural engineer/designer should consider all applicable codes and authoritative sources where needed. If analyses by the structural engineer/designer result in less critical details than are provided herein as minimums, the minimums presented herein should be adopted. It is considered likely that some, more restrictive details will be required.

If the structural engineer/designer has any questions or requires further assistance, they should not hesitate to call or otherwise transmit their requests to GSI. In order to mitigate potential distress, the foundation and/or improvement's designer should confirm to GSI and the governing agency, in writing, that the proposed foundations and/or improvements can tolerate the amount of differential settlement and/or expansion characteristics and other design criteria specified herein.

PLAN REVIEW

Final project plans (grading, precise grading, foundation, retaining wall, landscaping, etc.), should be reviewed by this office prior to construction, so that construction is in accordance with the conclusions and recommendations of this report. Based on our review, supplemental recommendations and/or further geotechnical studies may be warranted.

LIMITATIONS

The materials encountered on the project site and utilized for our analysis are believed representative of the area; however, soil and bedrock materials vary in character between excavations and natural outcrops or conditions exposed during mass grading. Site conditions may vary due to seasonal changes or other factors.

Inasmuch as our study is based upon our review and engineering analyses and laboratory data, the conclusions and recommendations are professional opinions. These opinions have been derived in accordance with current standards of practice, and no warranty, either express or implied, is given. Standards of practice are subject to change with time. GSI assumes no responsibility or liability for work or testing performed by others, or their inaction; or work performed when GSI is not requested to be onsite, to evaluate if our recommendations have been properly implemented. Use of this report constitutes an agreement and consent by the user to all the limitations outlined above, notwithstanding any other agreements that may be in place. In addition, this report may be subject to review by the controlling authorities. Thus, this report brings to completion our scope of services for this portion of the project. All samples will be disposed of after 30 days, unless specifically requested by the Client, in writing.

APPENDIX A

REFERENCES

APPENDIX A

REFERENCES

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APPENDIX B

TEST PIT AND BORING LOGS

UNIFIED SOIL CLASSIFICATION SYSTEM				
Major Divisions			**Group Symbols**	**Typical Names**
Coarse-Grained Soils More than 50% retained on No. 200 sieve	Gravels 50% or more of coarse fraction retained on No. 4 sieve	Clean Gravels	GW	Well-graded gravels and gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines
		Gravel with	GM	Silty gravels gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	Sands more than 50% of coarse fraction passes No. 4 sieve	Clean Sands	SW	Well-graded sands and gravelly sands, little or no fines
			SP	Poorly graded sands and gravelly sands, little or no fines
		Sands with Fines	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
Fine-Grained Soils 50% or more passes No. 200 sieve	Silts and Clays Liquid limit 50% or less		ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and organic silty clays of low plasticity
	Silts and Clays Liquid limit greater than 50%		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity
Highly Organic Soils			PT	Peat, mucic, and other highly organic soils

CONSISTENCY OR RELATIVE DENSITY

CRITERIA

Standard Penetration Test

Penetration Resistance N (blows/ft)	Relative Density
0 - 4	Very loose
4 - 10	Loose
10 - 30	Medium
30 - 50	Dense
> 50	Very dense

Standard Penetration Test

Penetration Resistance N (blows/ft)	Consistency	Unconfined Compressive Strength (tons/ft^2)
<2	Very Soft	<0.25
2 - 4	Soft	0.25 - .050
4 - 8	Medium	0.50 - 1.00
8 - 15	Stiff	1.00 - 2.00
15 - 30	Very Stiff	2.00 - 4.00
>30	Hard	>4.00

	3"	3/4"		#4	#10	#40	#200 U.S. Standard Sieve
Unified Soil Classification	Cobbles	Gravel		Sand			Silt or Clay
		coarse	fine	coarse	medium	fine	

MOISTURE CONDITIONS

Dry	Absence of moisture: dusty, dry to the touch
Slightly Moist	Below optimum moisture content for compaction
Moist	Near optimum moisture content
Very Moist	Above optimum moisture content
Wet	Visible free water; below water table

MATERIAL QUANTITY

trace	0 - 5 %
few	5 - 10 %
little	10 - 25 %
some	25 - 45 %

OTHER SYMBOLS

C	Core Sample
S	SPT Sample
B	Bulk Sample
▼	Groundwater
Qp	Pocket Penetrometer

BASIC LOG FORMAT:
Group name, Group symbol, (grain size), color, moisture, consistency or relative density. Additional comments: odor, presence of roots, mica, gypsum, coarse grained particles, etc.

EXAMPLE:
Sand (SP), fine to medium grained, brown, moist, loose, trace silt, little fine gravel, few cobbles up to 4" in size, some hair roots and rootlets.

GeoSoils, Inc.

W.O. 5654-A2-SC
Valley Center View Properties, LP
APN 188-231-34, Valley Center
Logged By: RB
March 5, 2008

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV.	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-1		0-1½	SM				**QUATERNARY COLLUVIUM:** SILTY SAND, dark brown, wet, loose; trace organics, trace rock fragments.
		1½-4	SC/CL				**HIGHLY WEATHERED PLUTONIC BEDROCK:** CLAYEY SAND/SANDY CLAY, brown, moist, medium dense/stiff.
		4-7	SM				**PLUTONIC BEDROCK:** DECOMPOSED GRANITE, brown to gray, moist, dense; breaks to SILTY SAND upon excavation.
							Total Depth = 7' No Groundwater/Caving Encountered Backfilled 3-5-2008
TP-2		0-2½	SM				**QUATERNARY COLLUVIUM:** SILTY SAND, dark brown, wet, loose; trace organics.
		2½-4	SM/ML				**HIGHLY WEATHERED PLUTONIC BEDROCK:** SILTY SAND/ SANDY SILT, brown, moist, medium dense.
		4-4½	SM				**PLUTONIC BEDROCK:** DECOMPOSED GRANITE, gray to brown, moist, dense; massive, breaks to SILTY SAND upon excavation.
							Total Depth = 4½' No Groundwater/Caving Encountered Backfilled 3-5-2008

GeoSoils, Inc.

W.O. 5654-A2-SC
Valley Center View Properties, LP
APN 188-231-34, Valley Center
Logged By: RB
March 6, 2008

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV.	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-3		0-2	SM				**QUATERNARY COLLUVIUM:** SILTY SAND, brown, moist, medium dense; trace angular pebble- to cobble-sized clasts.
		2-3½	SM				**PLUTONIC BEDROCK:** DECOMPOSED GRANITE, gray to brown, damp, very dense; massive breakage to SILTY SAND upon excavation.
							Total Depth = 3½' No Groundwater/Caving Encountered Backfilled 3-6-2008
TP-4		0-1	SM				**ARTIFICIAL FILL (UNDOCUMENTED):** SILTY SAND, brown, dry to damp, medium dense.
		1-1½	SM				**PLUTONIC BEDROCK:** DECOMPOSED GRANITE, gray to brown, dry, very dense; massive, breaks to SILTY SAND upon excavation.
							Total Depth = 1½' No Groundwater/Caving Encountered Backfilled 3-6-2008

GeoSoils, Inc.

W.O. 5654-A2-SC
Valley Center View Properties, LP
APN 188-231-34, Valley Center
Logged By: RB
January 19, 2009

LOG OF EXPLORATORY HAND AUGER

HAND AUGER NO.	ELEV.	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
HA-1		0-3	SM	Bulk @ 0-4½			**UNDOCUMENTED ARTIFICIAL FILL:** SILTY SAND, dark brown, dry becoming moist with depth, medium dense; abundant angular to sub angular pebbles and cobble-sized clasts.
		3-4½	SM				**PLUTONIC BEDROCK:** DECOMPOSED GRANITE, brownish gray, dry, medium dense becoming very dense with depth; breaks to SILTY SAND upon excavation.
							Practical Refusal @ 4½' No Groundwater/Caving Encountered Backfill 1-19-2009
HA-2		0-2	SM				**QUATERNARY COLLUVIUM:** SILTY SAND, grayish brown, dry becoming moist with depth; loose abundant angular to sub angular pebble-and cobble-sized clasts, porous.
		2-4	SM				**PLUTONIC BEDROCK:** DECOMPOSED GRANITE, brownish gray, dry, medium dense becoming very dense with depth; breaks to SILTY SAND upon excavation.
							Practical Refusal @ 4' No Groundwater/Caving Encountered Backfill 1-19-2009

APPENDIX C

EQFAULT, EQSEARCH, AND FRISKSP

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*    E Q F A U L T     *
*                      *
*    Version 3.00      *
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DETERMINISTIC ESTIMATION OF
PEAK ACCELERATION FROM DIGITIZED FAULTS

JOB NUMBER: 5654-A2-SC

DATE: 01-16-2009

JOB NAME: VALLEY CENTER VIEW PROPERTIES, L.P.

CALCULATION NAME: 5654-A2

FAULT-DATA-FILE NAME: C:\Program Files\EQFAULT1\CGSFLTE.DAT

SITE COORDINATES:
SITE LATITUDE: 33.2315
SITE LONGITUDE: 117.0319

SEARCH RADIUS: 62.14 mi

ATTENUATION RELATION: 13) Bozorgnia Campbell Niazi (1999) Hor.-Hard Rock-Cor.
UNCERTAINTY (M=Median, S=Sigma): S   Number of Sigmas: 1.0
DISTANCE MEASURE: cdist
SCOND: 1
Basement Depth: .00 km   Campbell SSR: 0   Campbell SHR: 1
COMPUTE PEAK HORIZONTAL ACCELERATION

FAULT-DATA FILE USED: C:\Program Files\EQFAULT1\CGSFLTE.DAT

MINIMUM DEPTH VALUE (km): 3.0

EQFAULT SUMMARY

DETERMINISTIC SITE PARAMETERS

Page 1

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT: MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
ELSINORE (JULIAN)	8.9(14.3)	7.1	0.398	X
ELSINORE (TEMECULA)	10.3(16.5)	6.8	0.288	IX
ROSE CANYON	22.3(35.9)	7.2	0.171	VIII
NEWPORT-INGLEWOOD (Offshore)	23.6(38.0)	7.1	0.151	VIII
EARTHQUAKE VALLEY	26.3(42.4)	6.5	0.089	VII
SAN JACINTO-ANZA	31.4(50.5)	7.2	0.120	VII
ELSINORE (GLEN IVY)	33.9(54.5)	6.8	0.084	VII
SAN JACINTO-COYOTE CREEK	34.1(54.9)	6.6	0.073	VII
SAN JACINTO-SAN JACINTO VALLEY	35.8(57.6)	6.9	0.085	VII
CORONADO BANK	37.3(60.1)	7.6	0.134	VIII
ELSINORE (COYOTE MOUNTAIN)	42.9(69.1)	6.8	0.065	VI
SAN JOAQUIN HILLS	45.2(72.7)	6.6	0.077	VII
SAN JACINTO - BORREGO	48.5(78.1)	6.6	0.050	VI
CHINO-CENTRAL AVE. (Elsinore)	51.6(83.0)	6.7	0.071	VI
PALOS VERDES	52.6(84.7)	7.3	0.075	VII
SAN ANDREAS - San Bernardino M-1	54.4(87.5)	7.5	0.084	VII
SAN ANDREAS - Whole M-1a	54.4(87.5)	8.0	0.122	VII
SAN ANDREAS - SB-Coach. M-1b-2	54.4(87.5)	7.7	0.097	VII
SAN ANDREAS - SB-Coach. M-2b	54.4(87.5)	7.7	0.097	VII
SAN JACINTO-SAN BERNARDINO	55.6(89.5)	6.7	0.046	VI
WHITTIER	55.7(89.6)	6.8	0.050	VI
SAN ANDREAS - Coachella M-1c-5	57.7(92.9)	7.2	0.063	VI
NEWPORT-INGLEWOOD (L.A.Basin)	57.7(92.9)	7.1	0.059	VI
PINTO MOUNTAIN	59.7(96.1)	7.2	0.061	VI
BURNT MTN.	61.1(98.4)	6.5	0.037	V

****

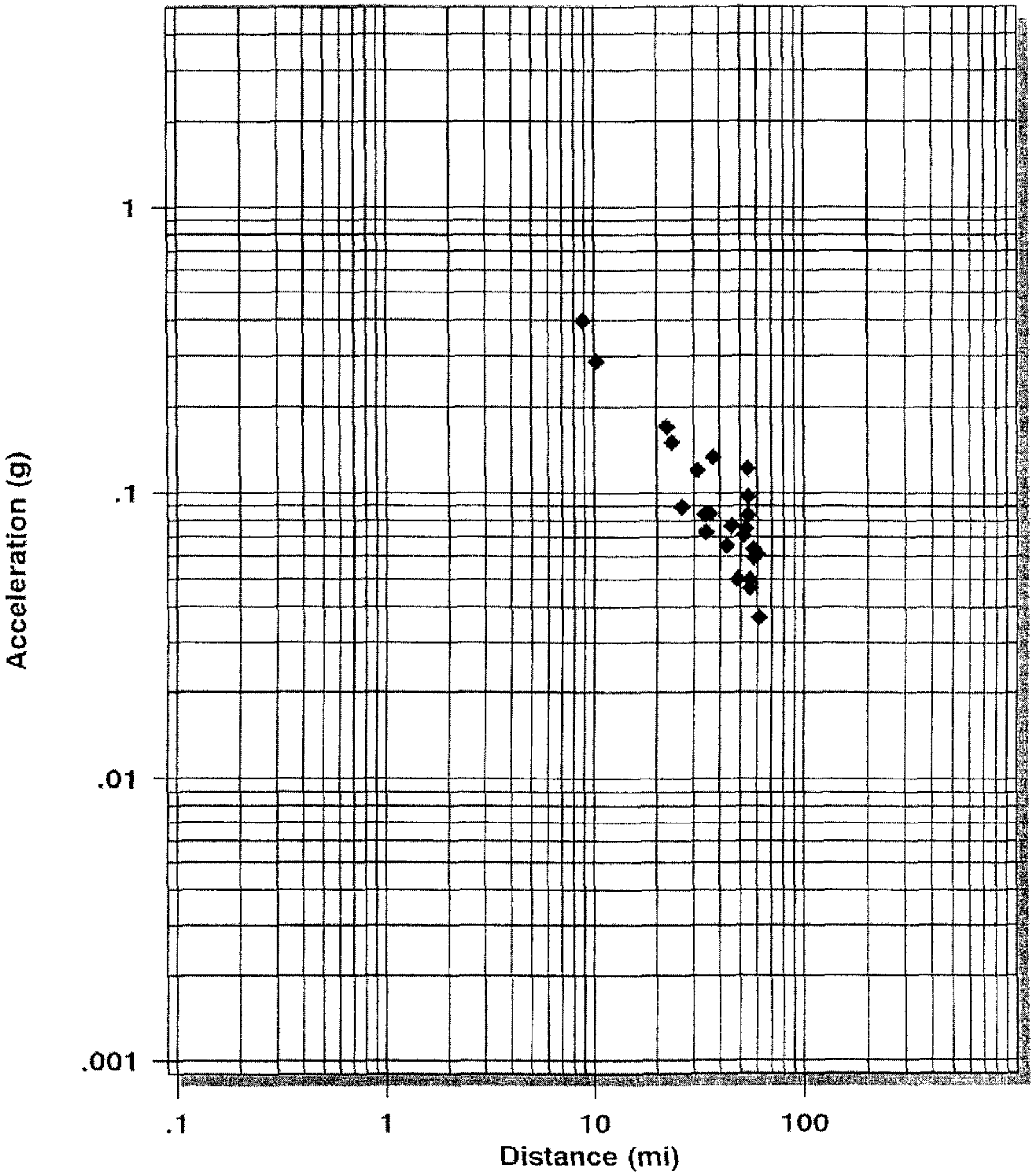
-END OF SEARCH- 25 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE ELSINORE (JULIAN) FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 8.9 MILES (14.3 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.3982 g

MAXIMUM EARTHQUAKES

VALLEY CENTER VIEW PROPERTIES, L.P.



W.O. 5654-A2-SC

Plate C-3

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*     Version 3.00      *
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ESTIMATION OF
PEAK ACCELERATION FROM
CALIFORNIA EARTHQUAKE CATALOGS

JOB NUMBER: 5654-A2-SC

DATE: 01-16-2009

JOB NAME: VALLEY CENTER VIEW PROPERTIES, L.P.

EARTHQUAKE-CATALOG-FILE NAME: ALLQUAKE.DAT

SITE COORDINATES:
SITE LATITUDE: 33.2315
SITE LONGITUDE: 117.0319

SEARCH DATES:
START DATE: 1800
END DATE: 2008

SEARCH RADIUS:
62.1 mi
100.0 km

ATTENUATION RELATION: 13) Bozorgnia Campbell Niazi (1999) Hor.-Hard Rock-Cor.
UNCERTAINTY (M=Median, S=Sigma): S   Number of Sigmas: 1.0
ASSUMED SOURCE TYPE: SS [SS=Strike-slip, DS=Reverse-slip, BT=Blind-thrust]
SCOND: 0 Depth Source: A
Basement Depth: .00 km   Campbell SSR: 0   Campbell SHR: 1
COMPUTE PEAK HORIZONTAL ACCELERATION

MINIMUM DEPTH VALUE (km): 3.0

EARTHQUAKE SEARCH RESULTS

FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (UTC) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
MGI	33.0000	117.0000	09/21/1856	730 0.0	0.0	5.00	0.058	VI	16.1(25.9)
DMG	33.2000	116.7000	01/01/1920	235 0.0	0.0	5.00	0.048	VI	19.3(31.0)
DMG	33.0000	117.3000	11/22/1800	2130 0.0	0.0	6.50	0.106	VII	22.3(35.8)
MGI	33.2000	116.6000	10/12/1920	1748 0.0	0.0	5.30	0.044	VI	25.0(40.3)
MGI	32.8000	117.1000	05/25/1803	0 0 0.0	0.0	5.00	0.031	V	30.0(48.4)
DMG	32.8000	116.8000	10/23/1894	23 3 0.0	0.0	5.70	0.043	VI	32.7(52.6)
GSP	33.5290	116.5720	06/12/2005	154146.5	14.0	5.20	0.031	V	33.5(54.0)
DMG	33.7100	116.9250	09/23/1963	144152.6	16.5	5.00	0.027	V	33.6(54.1)
PAS	33.5010	116.5130	02/25/1980	104738.5	13.6	5.50	0.035	V	35.2(56.7)
GSP	33.5080	116.5140	10/31/2001	075616.6	15.0	5.10	0.027	V	35.4(57.0)
DMG	33.5000	116.5000	09/30/1916	211 0.0	0.0	5.00	0.026	V	35.8(57.7)
DMG	33.7500	117.0000	04/21/1918	223225.0	0.0	6.80	0.079	VII	35.8(57.7)
DMG	33.7500	117.0000	06/06/1918	2232 0.0	0.0	5.00	0.026	V	35.8(57.7)
DMG	32.7000	117.2000	05/27/1862	20 0 0.0	0.0	5.90	0.041	V	38.0(61.1)
DMG	33.0000	116.4330	06/04/1940	1035 8.3	0.0	5.10	0.025	V	38.1(61.4)
DMG	33.7000	117.4000	05/15/1910	1547 0.0	0.0	6.00	0.043	VI	38.7(62.2)
DMG	33.7000	117.4000	05/13/1910	620 0.0	0.0	5.00	0.024	IV	38.7(62.2)
DMG	33.7000	117.4000	04/11/1910	757 0.0	0.0	5.00	0.024	IV	38.7(62.2)
DMG	33.8000	117.0000	12/25/1899	1225 0.0	0.0	6.40	0.055	VI	39.3(63.2)
T-A	32.6700	117.1700	05/24/1865	0 0 0.0	0.0	5.00	0.023	IV	39.6(63.7)
T-A	32.6700	117.1700	12/00/1856	0 0 0.0	0.0	5.00	0.023	IV	39.6(63.7)
T-A	32.6700	117.1700	10/21/1862	0 0 0.0	0.0	5.00	0.023	IV	39.6(63.7)
DMG	33.3430	116.3460	04/28/1969	232042.9	20.0	5.80	0.036	V	40.3(64.9)
DMG	33.6990	117.5110	05/31/1938	83455.4	10.0	5.50	0.029	V	42.5(68.3)
DMG	33.4000	116.3000	02/09/1890	12 6 0.0	0.0	6.30	0.046	VI	43.8(70.5)
DMG	33.4080	116.2610	03/25/1937	1649 1.8	10.0	6.00	0.036	V	46.1(74.2)
DMG	33.9000	117.2000	12/19/1880	0 0 0.0	0.0	6.00	0.035	V	47.2(75.9)
DMG	33.2000	116.2000	05/28/1892	1115 0.0	0.0	6.30	0.041	V	48.1(77.4)
DMG	33.2830	116.1830	03/19/1954	95556.0	0.0	5.00	0.018	IV	49.1(79.1)
DMG	33.2830	116.1830	03/23/1954	41450.0	0.0	5.10	0.019	IV	49.1(79.1)
DMG	33.2830	116.1830	03/19/1954	102117.0	0.0	5.50	0.025	V	49.1(79.1)
DMG	33.2830	116.1830	03/19/1954	95429.0	0.0	6.20	0.038	V	49.1(79.1)
DMG	33.9500	116.8500	09/28/1946	719 9.0	0.0	5.00	0.018	IV	50.7(81.6)
MGI	33.8000	117.6000	04/22/1918	2115 0.0	0.0	5.00	0.018	IV	51.1(82.2)
PAS	32.9710	117.8700	07/13/1986	1347 8.2	6.0	5.30	0.021	IV	51.7(83.2)
DMG	33.2170	116.1330	08/15/1945	175624.0	0.0	5.70	0.026	V	51.9(83.6)
DMG	33.1900	116.1290	04/09/1968	22859.1	11.1	6.40	0.041	V	52.2(84.1)
DMG	33.9760	116.7210	06/12/1944	104534.7	10.0	5.10	0.017	IV	54.4(87.6)
DMG	34.0000	117.2500	07/23/1923	73026.0	0.0	6.25	0.035	V	54.5(87.7)
DMG	33.9940	116.7120	06/12/1944	111636.0	10.0	5.30	0.019	IV	55.8(89.7)
DMG	32.7000	116.3000	02/24/1892	720 0.0	0.0	6.70	0.046	VI	56.1(90.2)
DMG	33.1130	116.0370	04/09/1968	3 353.5	5.0	5.20	0.017	IV	58.1(93.5)
PAS	33.9980	116.6060	07/08/1986	92044.5	11.7	5.60	0.022	IV	58.3(93.8)
DMG	33.2310	116.0040	05/26/1957	155933.6	15.1	5.00	0.015	IV	59.4(95.5)
MGI	34.0000	117.5000	12/16/1858	10 0 0.0	0.0	7.00	0.053	VI	59.5(95.7)
DMG	33.5750	117.9830	03/11/1933	518 4.0	0.0	5.20	0.017	IV	59.7(96.1)
DMG	33.6170	117.9670	03/11/1933	154 7.8	0.0	6.30	0.033	V	60.1(96.7)
DMG	33.9330	116.3830	12/04/1948	234317.0	0.0	6.50	0.037	V	61.1(98.4)
DMG	34.1000	116.8000	10/24/1935	1448 7.6	0.0	5.10	0.015	IV	61.4(98.8)

MGI |34.1000|117.3000|07/15/1905|2041 0.0| 0.0| 5.30| 0.017 | IV | 61.9(99.6)
****

-END OF SEARCH- 50 EARTHQUAKES FOUND WITHIN THE SPECIFIED SEARCH AREA.

TIME PERIOD OF SEARCH: 1800 TO 2008

LENGTH OF SEARCH TIME: 209 years

THE EARTHQUAKE CLOSEST TO THE SITE IS ABOUT 16.1 MILES (25.9 km) AWAY.

LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 7.0

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.106 g

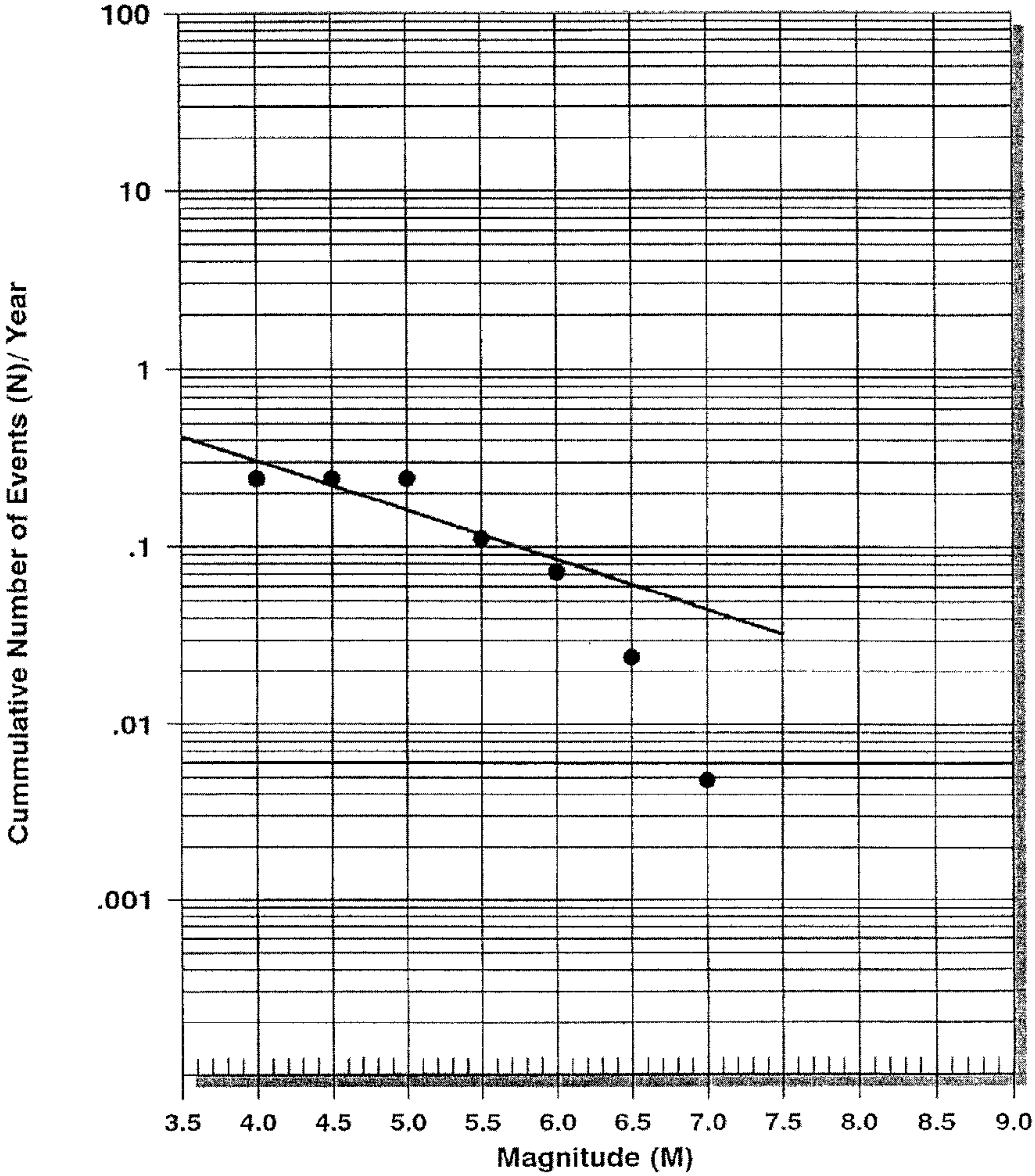
COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:
a-value= 0.590
b-value= 0.277
beta-value= 0.637

TABLE OF MAGNITUDES AND EXCEEDANCES:

Earthquake Magnitude	Number of Times Exceeded	Cumulative No. / Year
4.0	50	0.24038
4.5	50	0.24038
5.0	50	0.24038
5.5	23	0.11058
6.0	15	0.07212
6.5	5	0.02404
7.0	1	0.00481

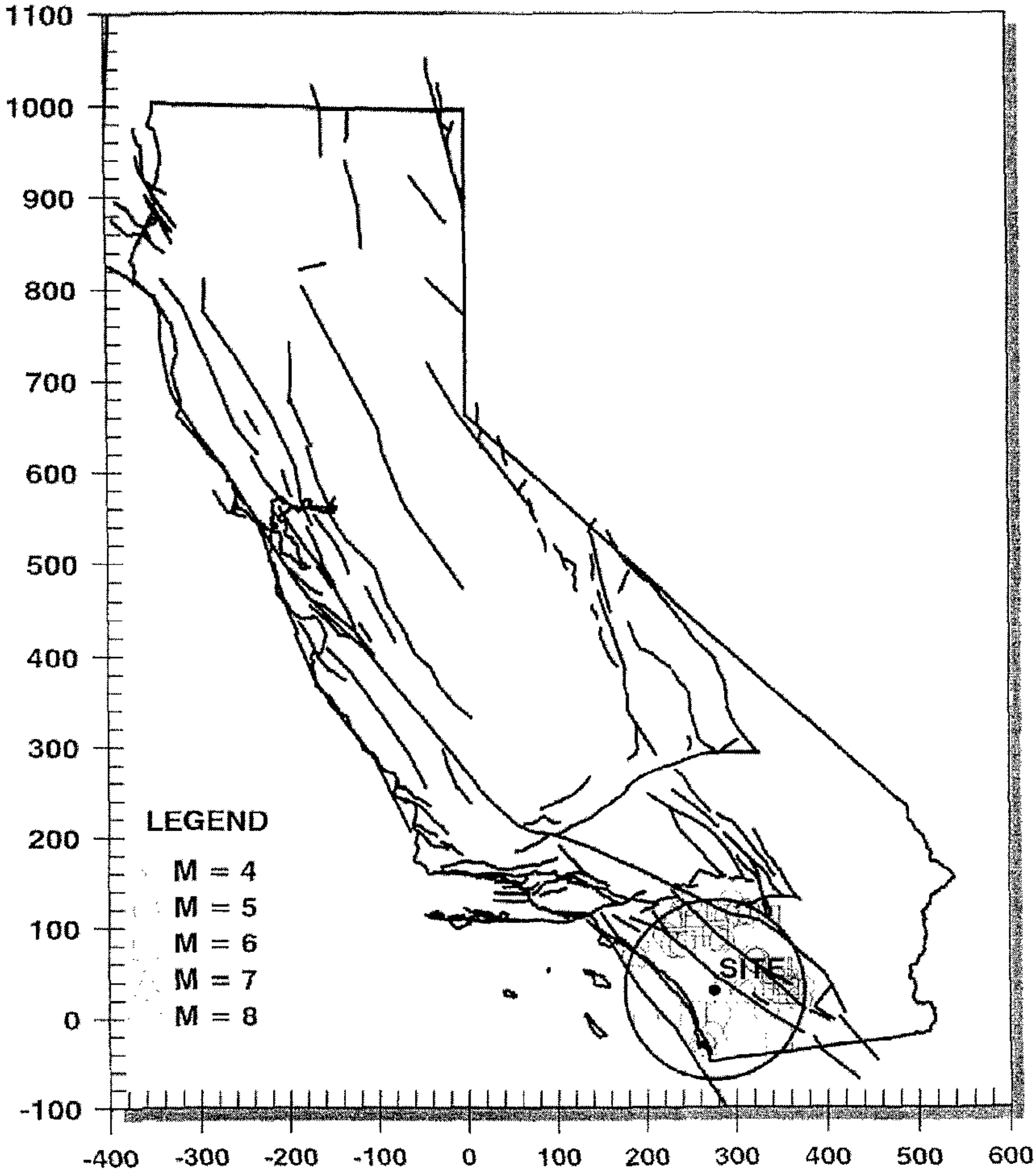
EARTHQUAKE RECURRENCE CURVE

VALLEY CENTER VIEW PROPERTIES, L.P.



EARTHQUAKE EPICENTER MAP

VALLEY CENTER VIEW PROPERTIES, L.P.

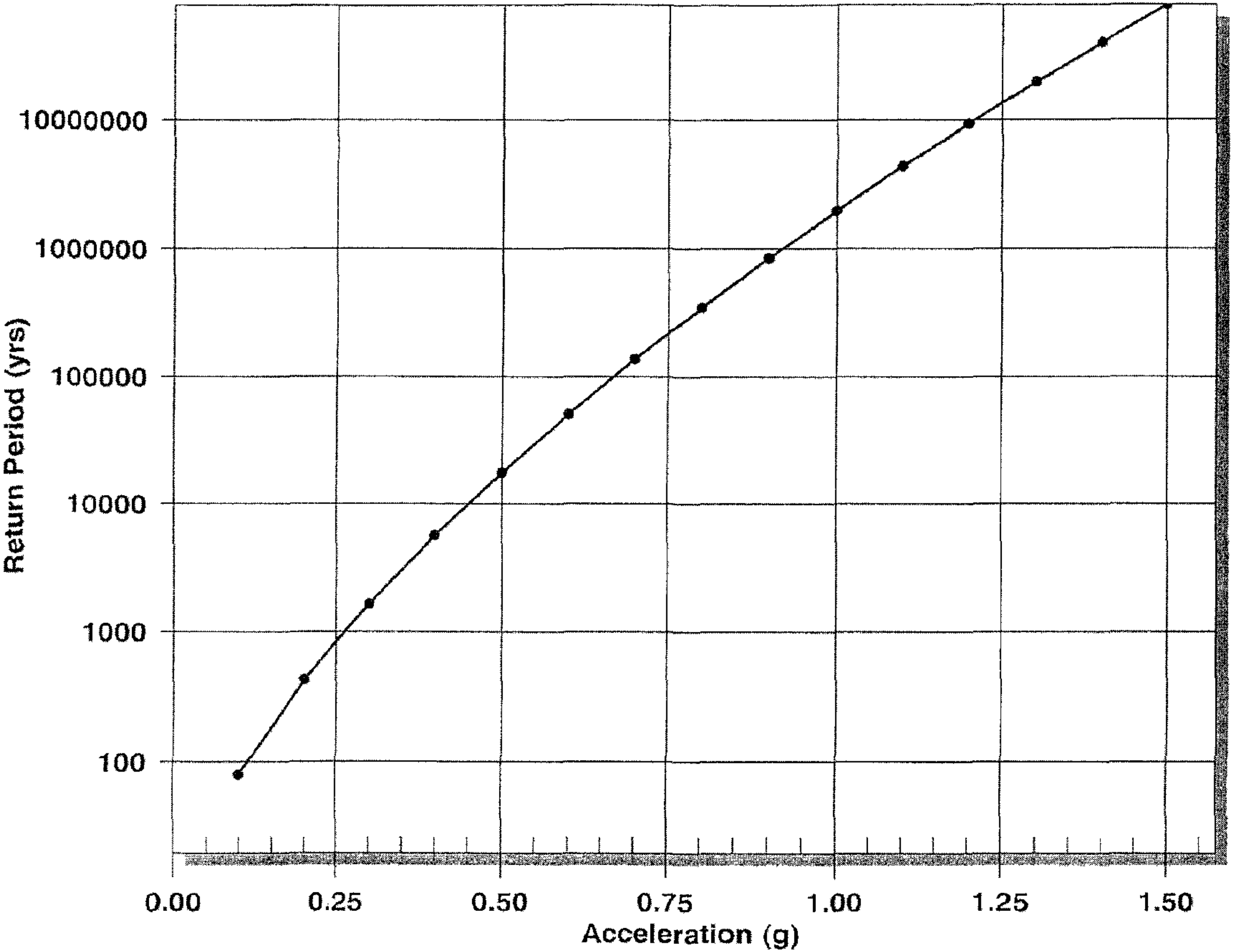


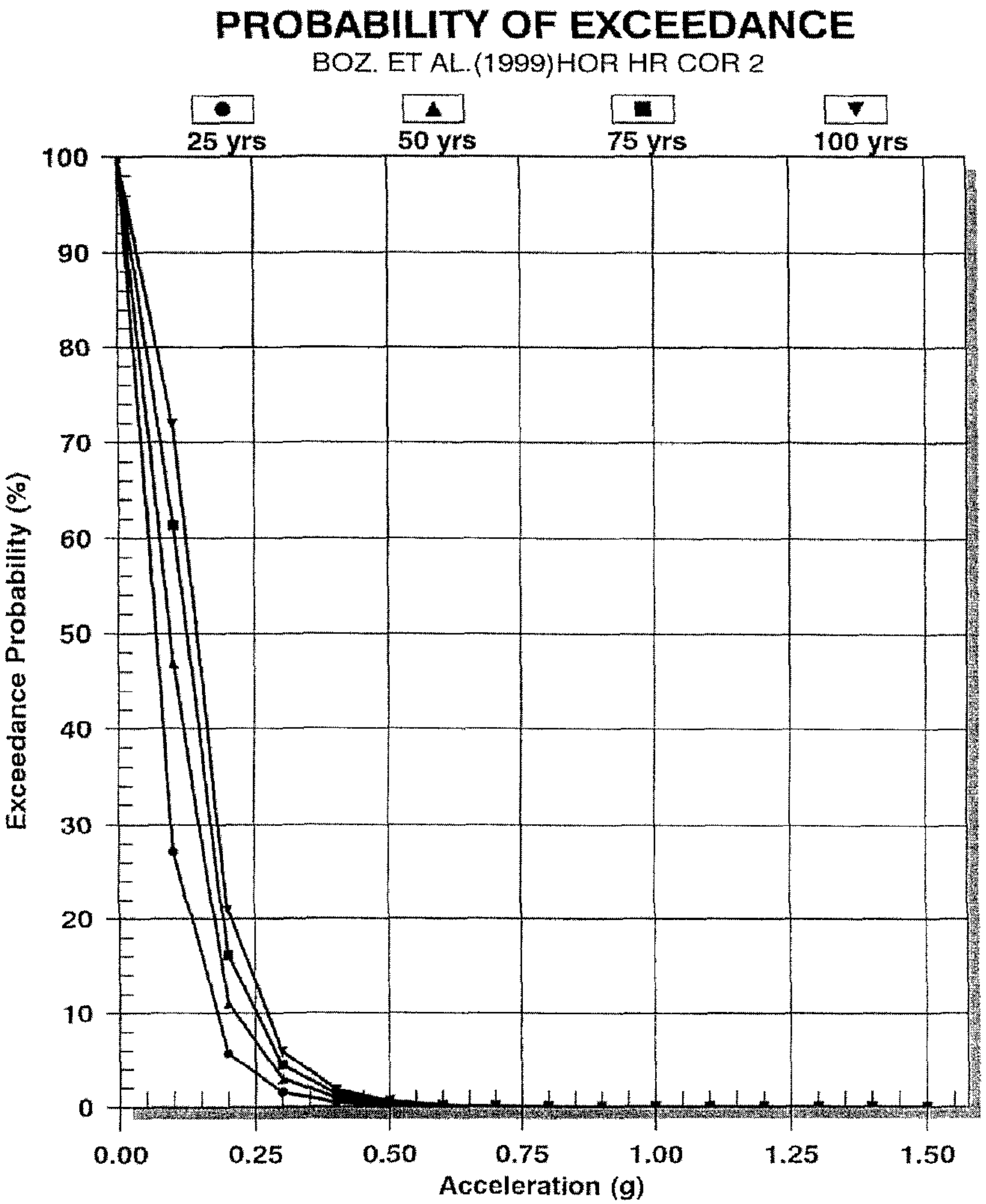
W.O. 5654-A2-SC

Plate C-8

RETURN PERIOD vs. ACCELERATION

BOZ. ET AL.(1999)HOR HR COR 2



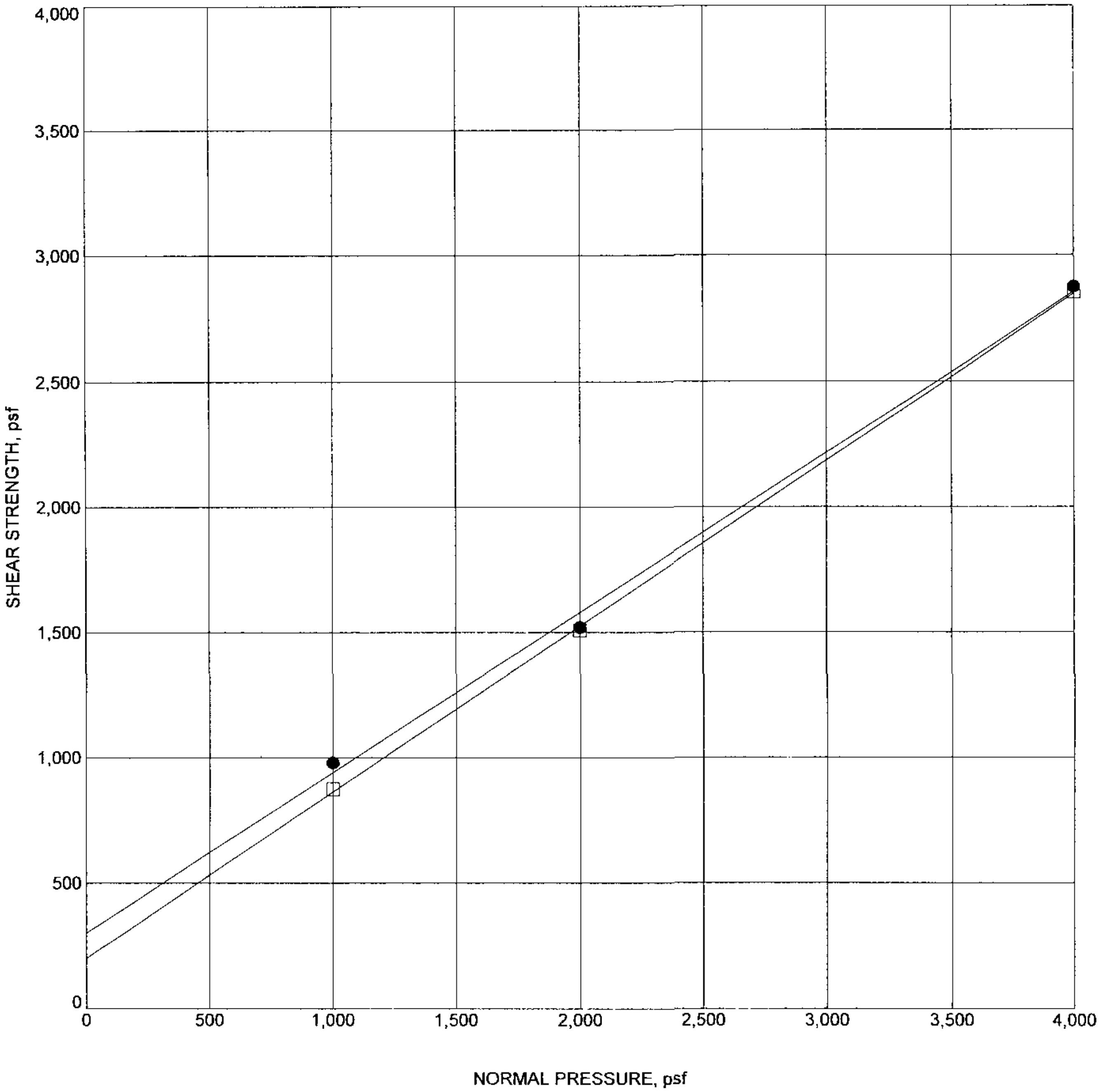


W.O. 5654-A2-SC

Plate C-10

APPENDIX D

LABORATORY DATA



	Sample	Depth/El.	Range	Classification	Primary/Residual	Sample Type	γ_d	MC%	c	ϕ
●	HA-1	0.0	0-4.5	Silty Sand	Primary Shear	Remolded	118.8	9.0	303	33
□	HA-1	0.0			Residual Shear	Remolded	118.8	9.0	202	33

Note: Sample Innundated prior to testing

GeoSoils, Inc.

GeoSoils, Inc.
5741 Palmer Way
Carlsbad, CA 92008
Telephone: (760) 438-3155
Fax: (760) 931-0915

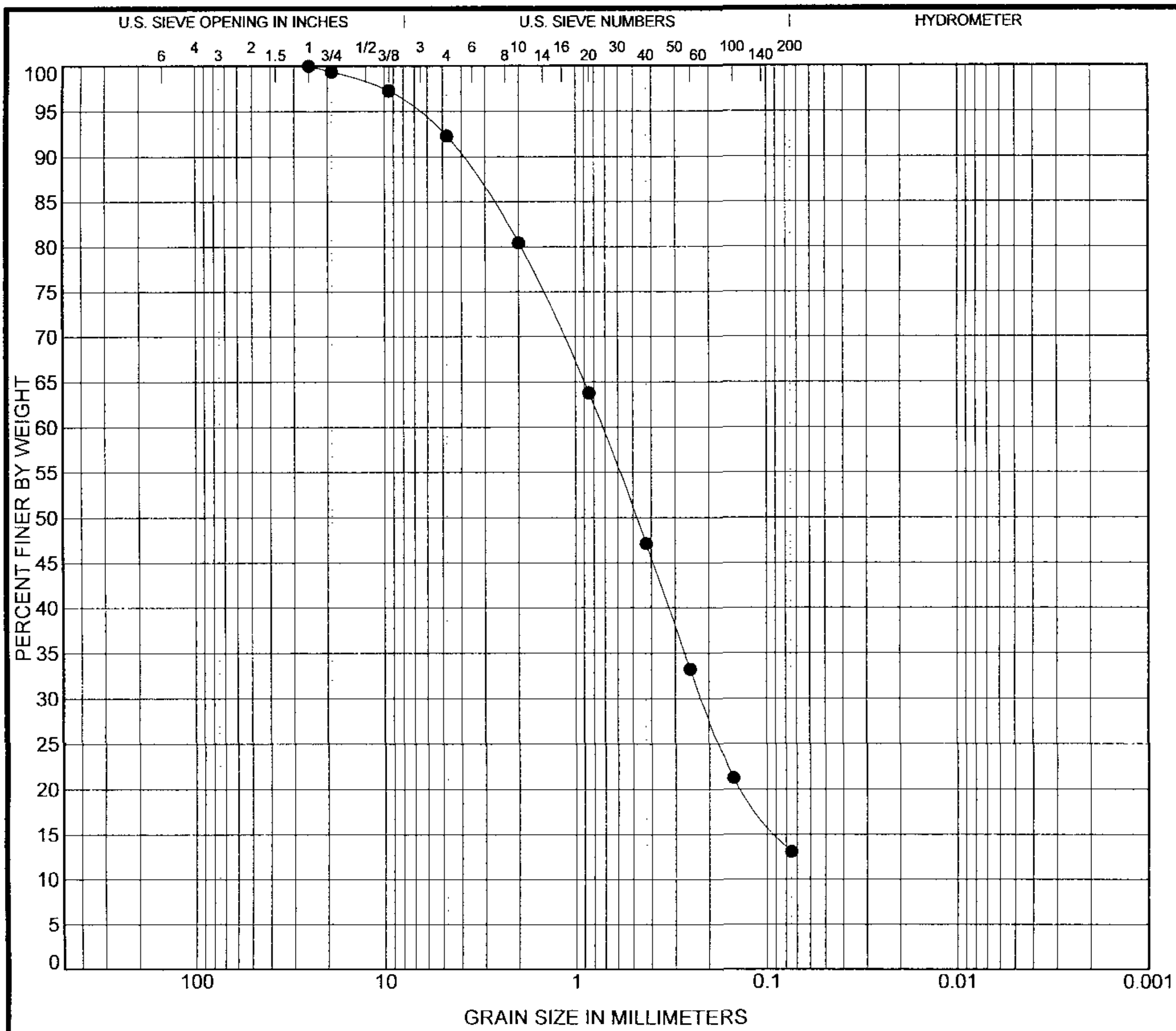
DIRECT SHEAR TEST

Project: VALLEY CENTER VIEW PROPERTIES L.P.

Number: 5654-A2-SC

Date: February 2009

Plate: D - 1



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Sample	Depth	Range	Visual Classification/USCS CLASSIFICATION	LL	PL	PI	Cc	Cu
●	HA-1	0.0	0-4.5	Silty Sand					

	Sample	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	HA-1	0.0	25	0.726	0.218		7.7	79.2	13.1	

US GRAIN SIZE 5654.GPJ US LAB.GDT 2/19/09

GeoSoils, Inc.

GeoSoils, Inc.
5741 Palmer Way
Carlsbad, CA 92008
Telephone: (760) 438-3155
Fax: (760) 931-0915

GRAIN SIZE DISTRIBUTION

Project: VALLEY CENTER VIEW PROPERTIES L.P.

Number: 5654-A2-SC

Date: February 2009

Plate: D - 2

Prime Testing, Inc.
38372 Innovation Ct Ste 102 Murrieta, CA 92563
ph (951) 894-2682 • fx (951) 894-2683

Work Order No.: 9A1200
Client: GeoSoils, Inc.
Project No.: 5654-A2-SC
Project Name: Valley Center View
Report Date: January 30, 2009

Laboratory Test(s) Results Summary

The subject soil sample was processed in accordance with California Test Method CTM 643 tested for pH (ASTM G 51), Soil Resistivity (ASTM G 57), Sulfate Content (ASTM D 516) and Chloride Content (ASTM D 512B). The test results follow:

Sample Identification	pH	As-Rec'd Resistivity (ohm-cm)	Saturated Resistivity (ohm-cm)	Sulfate Content (mg/kg)	Chloride Content (mg/kg)
HA-1 @ 0-4.5'	8.1	9,800	7,500	ND	ND

*ND=No Detection

We appreciate the opportunity to serve you. Please do not hesitate to contact us with any questions or clarifications regarding these results or procedures.

Ahmet K. Kaya, Laboratory Manager

ASTM INTERNATIONAL ORGANIZATIONAL MEMBER

W.O. 5654-A2-SC
PLATE D-3

TEST SPECIMEN		A	B	C	D
Compactor air pressure	PSI	350	350	350	
Water added	%	3.3	3.7	4.2	
Moisture at compaction	%	9.0	9.5	10.0	
Height of sample	IN	2.52	2.53	2.52	
Dry density	PCF	126.8	126.3	126.0	
R-Value by exudation		79	78	73	
R-Value by exudation, corrected		79	78	73	
Exudation pressure	PSI	510	316	228	
Stability thickness	FT	0.27	0.28	0.35	
Expansion pressure thickness	FT	0.50	0.37	0.27	

DESIGN CALCULATION DATA

Traffic index, assumed	5.0
Gravel equivalent factor, assumed	1.25
Expansion, stability equilibrium	0.3
R-Value by expansion	77
R-Value by exudation	78
R-Value at equilibrium	**77**

SAMPLE INFORMATION

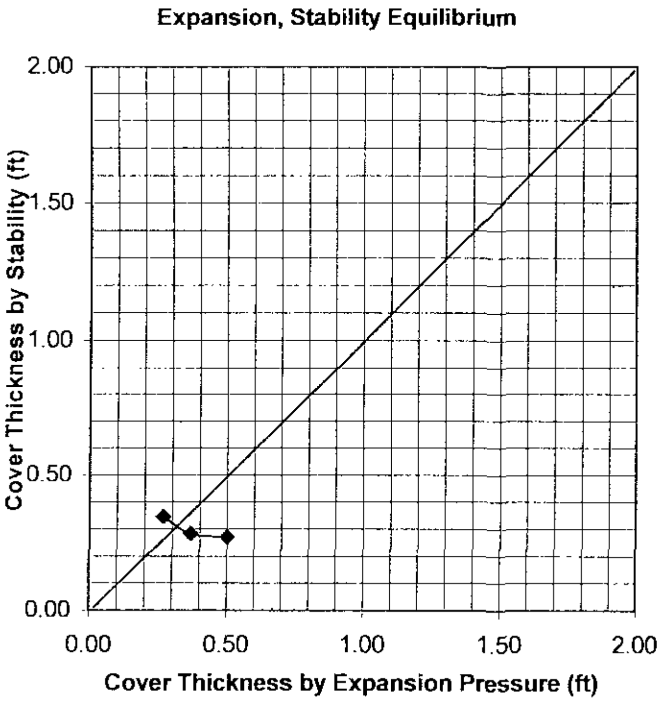
Sample Location: HA-1 @ 0-4.5

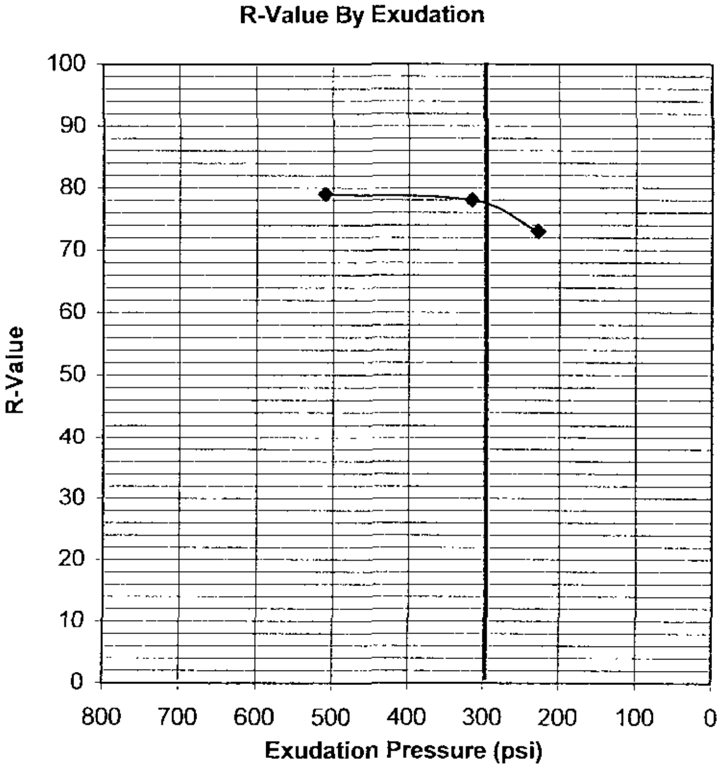
Sample Description: Yellow Brown Silty Sand

Notes:

1% Retained on 3/4 inch sieve

Test Method: Cal-Trans Test 301





GeoSoils, Inc.
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Carlsbad, CA 92008
Telephone: (760) 438-3155
Fax: (760) 931-0915

R - VALUE TEST RESULTS

Project: VALLEY CENTER VIEW PROPERTIES LP.

Number: 5654-A2-SC

Date: Feb-09 Plate: D - 4

APPENDIX E

GENERAL EARTHWORK, GRADING GUIDELINES AND PRELIMINARY CRITERIA

GENERAL EARTHWORK, GRADING GUIDELINES, AND PRELIMINARY CRITERIA

General

These guidelines present general procedures and requirements for earthwork and grading as shown on the approved grading plans, including preparation of areas to be filled, placement of fill, installation of subdrains, excavations, and appurtenant structures or flatwork. The recommendations contained in the geotechnical report are part of these earthwork and grading guidelines and would supercede the provisions contained hereafter in the case of conflict. Evaluations performed by the consultant during the course of grading may result in new or revised recommendations which could supercede these guidelines or the recommendations contained in the geotechnical report. Generalized details follow this text.

The contractor is responsible for the satisfactory completion of all earthwork in accordance with provisions of the project plans and specifications and latest adopted code. In the case of conflict, the most onerous provisions shall prevail. The project geotechnical engineer and engineering geologist (geotechnical consultant), and/or their representatives, should provide observation and testing services, and geotechnical consultation during the duration of the project.

EARTHWORK OBSERVATIONS AND TESTING

Geotechnical Consultant

Prior to the commencement of grading, a qualified geotechnical consultant (soil engineer and engineering geologist) should be employed for the purpose of observing earthwork procedures and testing the fills for general conformance with the recommendations of the geotechnical report(s), the approved grading plans, and applicable grading codes and ordinances.

The geotechnical consultant should provide testing and observation so that an evaluation may be made that the work is being accomplished as specified. It is the responsibility of the contractor to assist the consultants and keep them apprised of anticipated work schedules and changes, so that they may schedule their personnel accordingly.

All remedial removals, clean-outs, prepared ground to receive fill, key excavations, and subdrain installation should be observed and documented by the geotechnical consultant prior to placing any fill. It is the contractor's responsibility to notify the geotechnical consultant when such areas are ready for observation.

Laboratory and Field Tests

Maximum dry density tests to determine the degree of compaction should be performed in accordance with American Standard Testing Materials test method ASTM designation D-1557. Random or representative field compaction tests should be performed in accordance with test methods ASTM designation D-1556, D-2937 or D-2922, and D-3017, at intervals of approximately ±2 feet of fill height or approximately every 1,000 cubic yards placed. These criteria would vary depending on the soil conditions and the size of the project. The location and frequency of testing would be at the discretion of the geotechnical consultant.

Contractor's Responsibility

All clearing, site preparation, and earthwork performed on the project should be conducted by the contractor, with observation by a geotechnical consultant, and staged approval by the governing agencies, as applicable. It is the contractor's responsibility to prepare the ground surface to receive the fill, to the satisfaction of the geotechnical consultant, and to place, spread, moisture condition, mix, and compact the fill in accordance with the recommendations of the geotechnical consultant. The contractor should also remove all non-earth material considered unsatisfactory by the geotechnical consultant.

Notwithstanding the services provided by the geotechnical consultant, it is the sole responsibility of the contractor to provide adequate equipment and methods to accomplish the earthwork in strict accordance with applicable grading guidelines, latest adopted codes or agency ordinances, geotechnical report(s), and approved grading plans. Sufficient watering apparatus and compaction equipment should be provided by the contractor with due consideration for the fill material, rate of placement, and climatic conditions. If, in the opinion of the geotechnical consultant, unsatisfactory conditions such as questionable weather, excessive oversized rock or deleterious material, insufficient support equipment, etc., are resulting in a quality of work that is not acceptable, the consultant will inform the contractor, and the contractor is expected to rectify the conditions, and if necessary, stop work until conditions are satisfactory.

During construction, the contractor shall properly grade all surfaces to maintain good drainage and prevent ponding of water. The contractor shall take remedial measures to control surface water and to prevent erosion of graded areas until such time as permanent drainage and erosion control measures have been installed.

SITE PREPARATION

All major vegetation, including brush, trees, thick grasses, organic debris, and other deleterious material, should be removed and disposed of off-site. These removals must be concluded prior to placing fill. In-place existing fill, soil, alluvium, colluvium, or rock materials, as evaluated by the geotechnical consultant as being unsuitable, should be removed prior to any fill placement. Depending upon the soil conditions, these materials may be reused as compacted fills. Any materials incorporated as part of the compacted fills should be approved by the geotechnical consultant.

Any underground structures such as cesspools, cisterns, mining shafts, tunnels, septic tanks, wells, pipelines, or other structures not located prior to grading, are to be removed or treated in a manner recommended by the geotechnical consultant. Soft, dry, spongy, highly fractured, or otherwise unsuitable ground, extending to such a depth that surface processing cannot adequately improve the condition, should be overexcavated down to firm ground and approved by the geotechnical consultant before compaction and filling operations continue. Overexcavated and processed soils, which have been properly mixed and moisture conditioned, should be re-compacted to the minimum relative compaction as specified in these guidelines.

Existing ground, which is determined to be satisfactory for support of the fills, should be scarified (ripped) to a minimum depth of 6 to 8 inches, or as directed by the geotechnical consultant. After the scarified ground is brought to optimum moisture content, or greater and mixed, the materials should be compacted as specified herein. If the scarified zone is greater than 6 to 8 inches in depth, it may be necessary to remove the excess and place the material in lifts restricted to about 6 to 8 inches in compacted thickness.

Existing ground which is not satisfactory to support compacted fill should be overexcavated as required in the geotechnical report, or by the on-site geotechnical consultant. Scarification, disc harrowing, or other acceptable forms of mixing should continue until the soils are broken down and free of large lumps or clods, until the working surface is reasonably uniform and free from ruts, hollows, hummocks, mounds, or other uneven features, which would inhibit compaction as described previously.

Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical [h:v]), the ground should be stepped or benched. The lowest bench, which will act as a key, should be a minimum of 15 feet wide and should be at least 2 feet deep into firm material, and approved by the geotechnical consultant. In fill-over-cut slope conditions, the recommended minimum width of the lowest bench or key is also 15 feet, with the key founded on firm material, as designated by the geotechnical consultant. As a general rule, unless specifically recommended otherwise by the geotechnical consultant, the minimum width of fill keys should be equal to ½ the height of the slope.

Standard benching is generally 4 feet (minimum) vertically, exposing firm, acceptable material. Benching may be used to remove unsuitable materials, although it is understood that the vertical height of the bench may exceed 4 feet. Pre-stripping may be considered for unsuitable materials in excess of 4 feet in thickness.

All areas to receive fill, including processed areas, removal areas, and the toes of fill benches, should be observed and approved by the geotechnical consultant prior to placement of fill. Fills may then be properly placed and compacted until design grades (elevations) are attained.

COMPACTED FILLS

Any earth materials imported or excavated on the property may be utilized in the fill provided that each material has been evaluated to be suitable by the geotechnical consultant. These materials should be free of roots, tree branches, other organic matter, or other deleterious materials. All unsuitable materials should be removed from the fill as directed by the geotechnical consultant. Soils of poor gradation, undesirable expansion potential, or substandard strength characteristics may be designated by the consultant as unsuitable and may require blending with other soils to serve as a satisfactory fill material.

Fill materials derived from benching operations should be dispersed throughout the fill area and blended with other approved material. Benching operations should not result in the benched material being placed only within a single equipment width away from the fill/bedrock contact.

Oversized materials defined as rock, or other irreducible materials, with a maximum dimension greater than 12 inches, should not be buried or placed in fills unless the location of materials and disposal methods are specifically approved by the geotechnical consultant. Oversized material should be taken offsite, or placed in accordance with recommendations of the geotechnical consultant in areas designated as suitable for rock disposal. GSI anticipates that soils to be utilized as fill material for the subject project may contain some rock. Appropriately, the need for rock disposal may be necessary during grading operations on the site. From a geotechnical standpoint, the depth of any rocks, rock fills, or rock blankets, should be a sufficient distance from finish grade. This depth is generally the same as any overexcavation due to cut-fill transitions in hard rock areas, and generally facilitates the excavation of structural footings and substructures. Should deeper excavations be proposed (i.e., deepened footings, utility trenching, swimming pools, spas, etc.), the developer may consider increasing the hold-down depth of any rocky fills to be placed, as appropriate. In addition, some agencies/jurisdictions mandate a specific hold-down depth for oversize materials placed in fills. The hold-down depth, and potential to encounter oversize rock, both within fills, and occurring in cut or natural areas, would need to be disclosed to all interested/affected parties. Once approved by the governing agency, the hold-down depth for oversized rock (i.e., greater than 12 inches) in fills on this project is provided as 10 feet, unless specified differently in the text of this report. The governing agency may require that these materials need to be deeper, crushed, or reduced to less than 12 inches in maximum dimension, at their discretion.

To facilitate future trenching, rock (or oversized material), should not be placed within the hold-down depth feet from finish grade, the range of foundation excavations, future utilities, or underground construction unless specifically approved by the governing agency, the geotechnical consultant, and/or the developer's representative.

If import material is required for grading, representative samples of the materials to be utilized as compacted fill should be analyzed in the laboratory by the geotechnical consultant to evaluate it's physical properties and suitability for use onsite. Such testing should be performed three (3) days prior to importation. If any material other than that previously tested is encountered during grading, an appropriate analysis of this material should be conducted by the geotechnical consultant as soon as possible.

Approved fill material should be placed in areas prepared to receive fill in near horizontal layers, that when compacted, should not exceed about 6 to 8 inches in thickness. The geotechnical consultant may approve thick lifts if testing indicates the grading procedures are such that adequate compaction is being achieved with lifts of greater thickness. Each layer should be spread evenly and blended to attain uniformity of material and moisture suitable for compaction.

Fill layers at a moisture content less than optimum should be watered and mixed, and wet fill layers should be aerated by scarification, or should be blended with drier material. Moisture conditioning, blending, and mixing of the fill layer should continue until the fill materials have a uniform moisture content at, or above, optimum moisture.

After each layer has been evenly spread, moisture conditioned, and mixed, it should be uniformly compacted to a minimum of 90 percent of the maximum density as evaluated by ASTM test designation D-1557, or as otherwise recommended by the geotechnical consultant. Compaction equipment should be adequately sized and should be specifically designed for soil compaction, or of proven reliability to efficiently achieve the specified degree of compaction.

Where tests indicate that the density of any layer of fill, or portion thereof, is below the required relative compaction, or improper moisture is in evidence, the particular layer or portion shall be re-worked until the required density and/or moisture content has been attained. No additional fill shall be placed in an area until the last placed lift of fill has been tested and found to meet the density and moisture requirements, and is approved by the geotechnical consultant.

In general, per the 1997 UBC and/or latest adopted version of the California Building Code (CBC), fill slopes should be designed and constructed at a gradient of 2:1 (h:v), or flatter. Compaction of slopes should be accomplished by over-building a minimum of 3 feet horizontally, and subsequently trimming back to the design slope configuration. Testing shall be performed as the fill is elevated to evaluate compaction as the fill core is being developed. Special efforts may be necessary to attain the specified compaction in the fill slope zone. Final slope shaping should be performed by trimming and removing loose materials with appropriate equipment. A final evaluation of fill slope compaction should be based on observation and/or testing of the finished slope face. Where compacted fill slopes are designed steeper than 2:1 (h:v), prior approval from the governing agency, specific material types, a higher minimum relative compaction, special reinforcement, and special grading procedures will be recommended.

If an alternative to over-building and cutting back the compacted fill slopes is selected, then special effort should be made to achieve the required compaction in the outer 10 feet of each lift of fill by undertaking the following:

1. An extra piece of equipment consisting of a heavy, short-shanked sheepsfoot should be used to roll (horizontal) parallel to the slopes continuously as fill is placed. The sheepsfoot roller should also be used to roll perpendicular to the slopes, and extend out over the slope to provide adequate compaction to the face of the slope.

2. Loose fill should not be spilled out over the face of the slope as each lift is compacted. Any loose fill spilled over a previously completed slope face should be trimmed off or be subject to re-rolling.

3. Field compaction tests will be made in the outer (horizontal) ± 2 to ± 8 feet of the slope at appropriate vertical intervals, subsequent to compaction operations.

4. After completion of the slope, the slope face should be shaped with a small tractor and then re-rolled with a sheepsfoot to achieve compaction to near the slope face. Subsequent to testing to evaluate compaction, the slopes should be grid-rolled to achieve compaction to the slope face. Final testing should be used to evaluate compaction after grid rolling.

5. Where testing indicates less than adequate compaction, the contractor will be responsible to rip, water, mix, and recompact the slope material as necessary to achieve compaction. Additional testing should be performed to evaluate compaction.

SUBDRAIN INSTALLATION

Subdrains should be installed in approved ground in accordance with the approximate alignment and details indicated by the geotechnical consultant. Subdrain locations or materials should not be changed or modified without approval of the geotechnical consultant. The geotechnical consultant may recommend and direct changes in subdrain line, grade, and drain material in the field, pending exposed conditions. The location of constructed subdrains, especially the outlets, should be recorded/surveyed by the project civil engineer. Drainage at the subdrain outlets should be provided by the project civil engineer.

EXCAVATIONS

Excavations and cut slopes should be examined during grading by the geotechnical consultant. If directed by the geotechnical consultant, further excavations or overexcavation and refilling of cut areas should be performed, and/or remedial grading of cut slopes should be performed. When fill-over-cut slopes are to be graded, unless otherwise approved, the cut portion of the slope should be observed by the geotechnical consultant prior to placement of materials for construction of the fill portion of the slope. The geotechnical consultant should observe all cut slopes, and should be notified by the contractor when excavation of cut slopes commence.

If, during the course of grading, unforeseen adverse or potentially adverse geologic conditions are encountered, the geotechnical consultant should investigate, evaluate, and make appropriate recommendations for mitigation of these conditions. The need for cut slope buttressing or stabilizing should be based on in-grading evaluation by the geotechnical consultant, whether anticipated or not.

Unless otherwise specified in geotechnical and geological report(s), no cut slopes should be excavated higher or steeper than that allowed by the ordinances of controlling governmental agencies. Additionally, short-term stability of temporary cut slopes is the contractor's responsibility.

Erosion control and drainage devices should be designed by the project civil engineer and should be constructed in compliance with the ordinances of the controlling governmental agencies, and/or in accordance with the recommendations of the geotechnical consultant.

COMPLETION

Observation, testing, and consultation by the geotechnical consultant should be conducted during the grading operations in order to state an opinion that all cut and fill areas are graded in accordance with the approved project specifications. After completion of grading, and after the geotechnical consultant has finished observations of the work, final reports should be submitted, and may be subject to review by the controlling governmental agencies. No further excavation or filling should be undertaken without prior notification of the geotechnical consultant or approved plans.

All finished cut and fill slopes should be protected from erosion and/or be planted in accordance with the project specifications and/or as recommended by a landscape architect. Such protection and/or planning should be undertaken as soon as practical after completion of grading.

JOB SAFETY

General

At GSI, getting the job done safely is of primary concern. The following is the company's safety considerations for use by all employees on multi-employer construction sites. On-ground personnel are at highest risk of injury, and possible fatality, on grading and construction projects. GSI recognizes that construction activities will vary on each site, and that site safety is the prime responsibility of the contractor; however, everyone must be safety conscious and responsible at all times. To achieve our goal of avoiding accidents, cooperation between the client, the contractor, and GSI personnel must be maintained.

In an effort to minimize risks associated with geotechnical testing and observation, the following precautions are to be implemented for the safety of field personnel on grading and construction projects:

Safety Meetings: GSI field personnel are directed to attend contractor's regularly scheduled and documented safety meetings.

Safety Vests: Safety vests are provided for, and are to be worn by GSI personnel, at all times, when they are working in the field.

Safety Flags: Two safety flags are provided to GSI field technicians; one is to be affixed to the vehicle when on site, the other is to be placed atop the spoil pile on all test pits.

Flashing Lights: All vehicles stationary in the grading area shall use rotating or flashing amber beacons, or strobe lights, on the vehicle during all field testing. While operating a vehicle in the grading area, the emergency flasher on the vehicle shall be activated.

In the event that the contractor's representative observes any of our personnel not following the above, we request that it be brought to the attention of our office.

Test Pits Location, Orientation, and Clearance

The technician is responsible for selecting test pit locations. A primary concern should be the technician's safety. Efforts will be made to coordinate locations with the grading contractor's authorized representative, and to select locations following or behind the established traffic pattern, preferably outside of current traffic. The contractor's authorized representative (supervisor, grade checker, dump man, operator, etc.) should direct excavation of the pit and safety during the test period. Of paramount concern should be the soil technician's safety, and obtaining enough tests to represent the fill.

Test pits should be excavated so that the spoil pile is placed away from oncoming traffic, whenever possible. The technician's vehicle is to be placed next to the test pit, opposite the spoil pile. This necessitates the fill be maintained in a driveable condition. Alternatively, the contractor may wish to park a piece of equipment in front of the test holes, particularly in small fill areas or those with limited access.

A zone of non-encroachment should be established for all test pits. No grading equipment should enter this zone during the testing procedure. The zone should extend approximately 50 feet outward from the center of the test pit. This zone is established for safety and to avoid excessive ground vibration, which typically decreases test results.

When taking slope tests, the technician should park the vehicle directly above or below the test location. If this is not possible, a prominent flag should be placed at the top of the slope. The contractor's representative should effectively keep all equipment at a safe operational distance (e.g., 50 feet) away from the slope during this testing.

The technician is directed to withdraw from the active portion of the fill as soon as possible following testing. The technician's vehicle should be parked at the perimeter of the fill in a highly visible location, well away from the equipment traffic pattern. The contractor should inform our personnel of all changes to haul roads, cut and fill areas or other factors that may affect site access and site safety.

In the event that the technician's safety is jeopardized or compromised as a result of the contractor's failure to comply with any of the above, the technician is required, by company policy, to immediately withdraw and notify his/her supervisor. The grading contractor's representative will be contacted in an effort to affect a solution. However, in the interim, no further testing will be performed until the situation is rectified. Any fill placed can be considered unacceptable and subject to reprocessing, recompaction, or removal.

In the event that the soil technician does not comply with the above or other established safety guidelines, we request that the contractor bring this to the technician's attention and notify this office. Effective communication and coordination between the contractor's representative and the soil technician is strongly encouraged in order to implement the above safety plan.

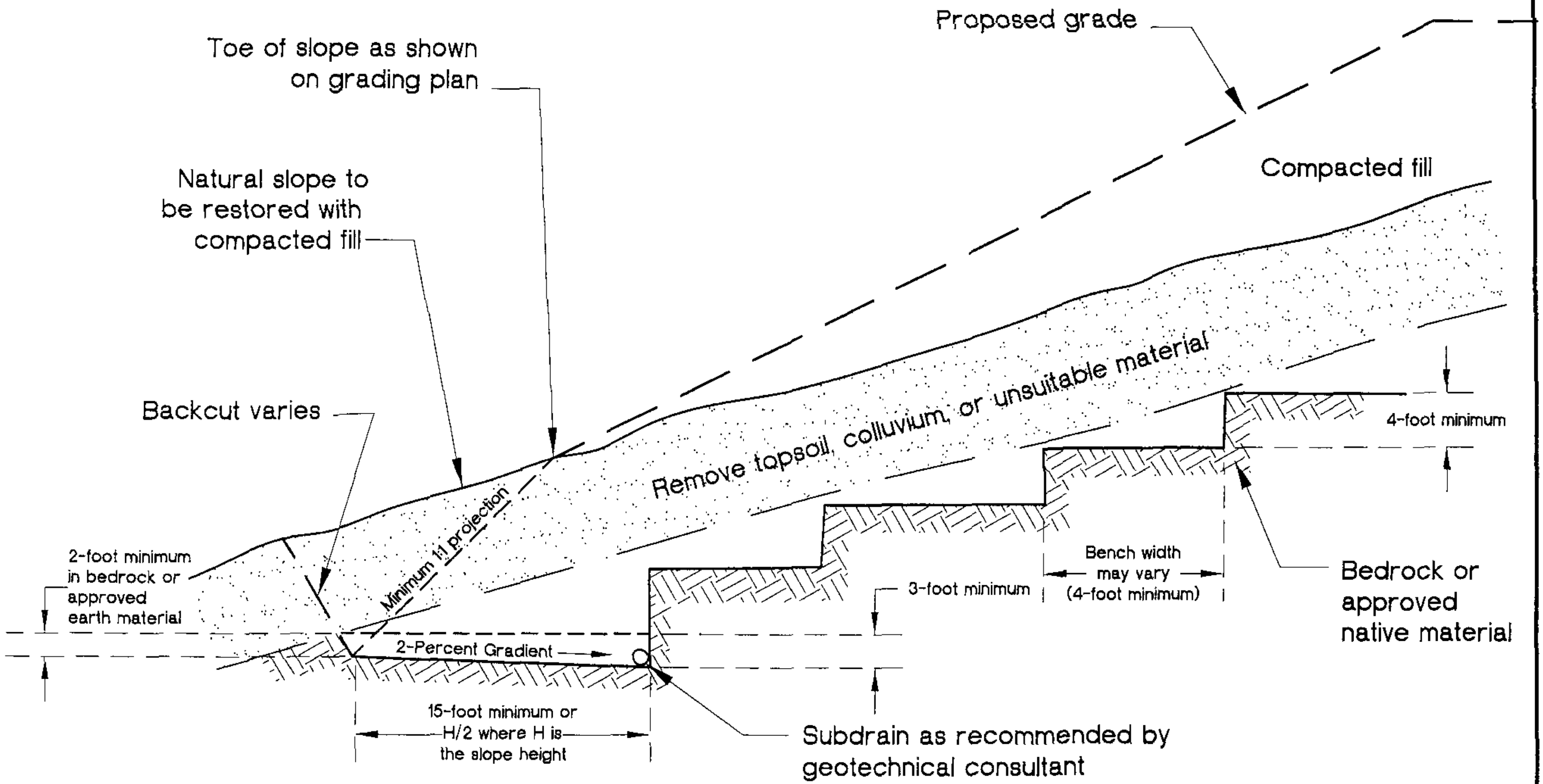
Trench and Vertical Excavation

It is the contractor's responsibility to provide safe access into trenches where compaction testing is needed. Our personnel are directed not to enter any excavation or vertical cut which: 1) is 5 feet or deeper unless shored or laid back; 2) displays any evidence of instability, has any loose rock or other debris which could fall into the trench; or 3) displays any other evidence of any unsafe conditions regardless of depth.

All trench excavations or vertical cuts in excess of 5 feet deep, which any person enters, should be shored or laid back. Trench access should be provided in accordance with Cal/OSHA and/or state and local standards. Our personnel are directed not to enter any trench by being lowered or "riding down" on the equipment.

If the contractor fails to provide safe access to trenches for compaction testing, our company policy requires that the soil technician withdraw and notify his/her supervisor. The contractor's representative will be contacted in an effort to affect a solution. All backfill not tested due to safety concerns or other reasons could be subject to reprocessing and/or removal.

If GSI personnel become aware of anyone working beneath an unsafe trench wall or vertical excavation, we have a legal obligation to put the contractor and owner/developer on notice to immediately correct the situation. If corrective steps are not taken, GSI then has an obligation to notify Cal/OSHA and/or the proper controlling authorities.

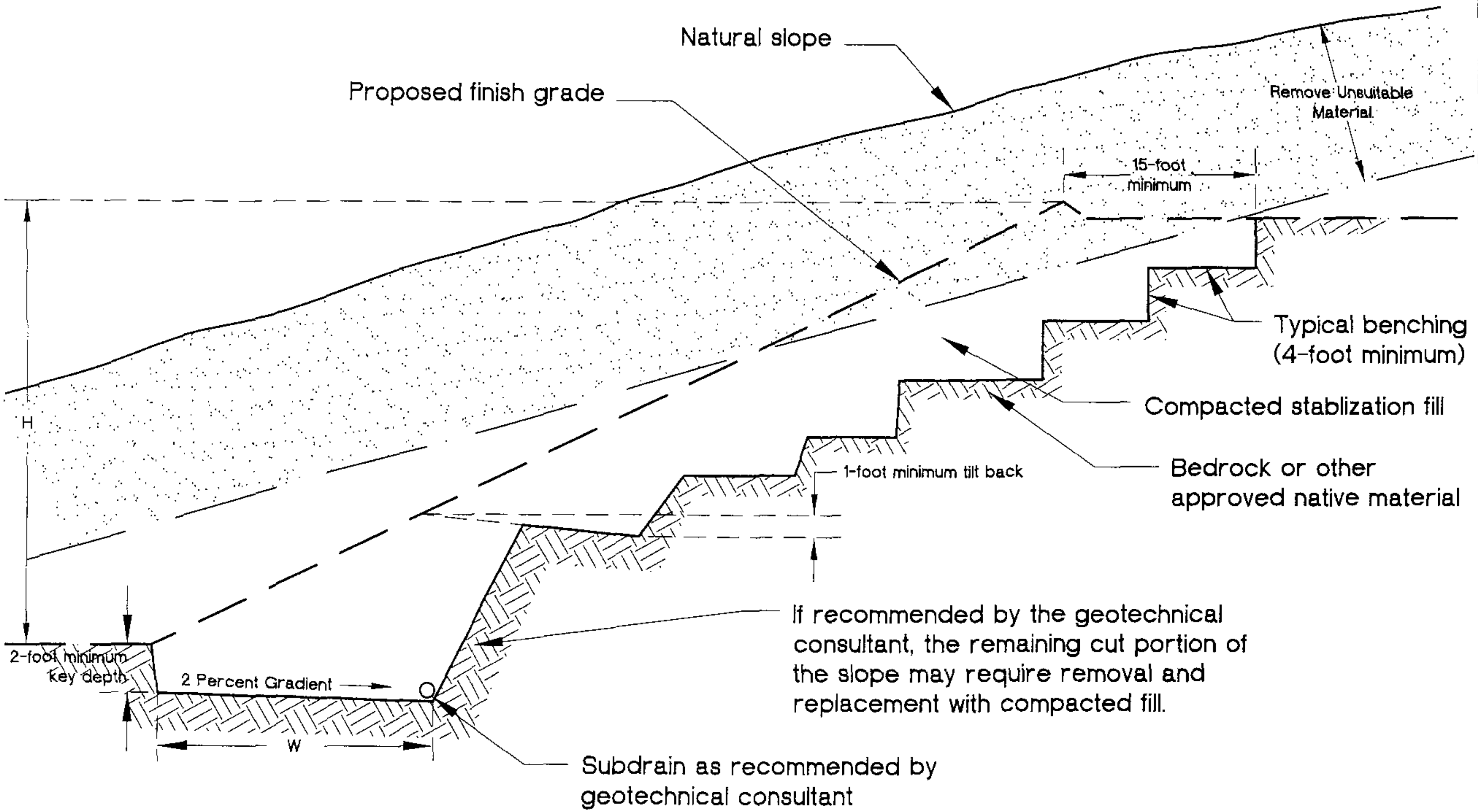


NOTES:

1. Where the natural slope approaches or exceeds the design slope ratio, special recommendations would be provided by the geotechnical consultant.

2. The need for and disposition of drains should be evaluated by the geotechnical consultant, based upon exposed conditions.

GeoSoils, Inc.	FILL OVER NATURAL (SIDEHILL FILL) DETAIL	Plate E-7

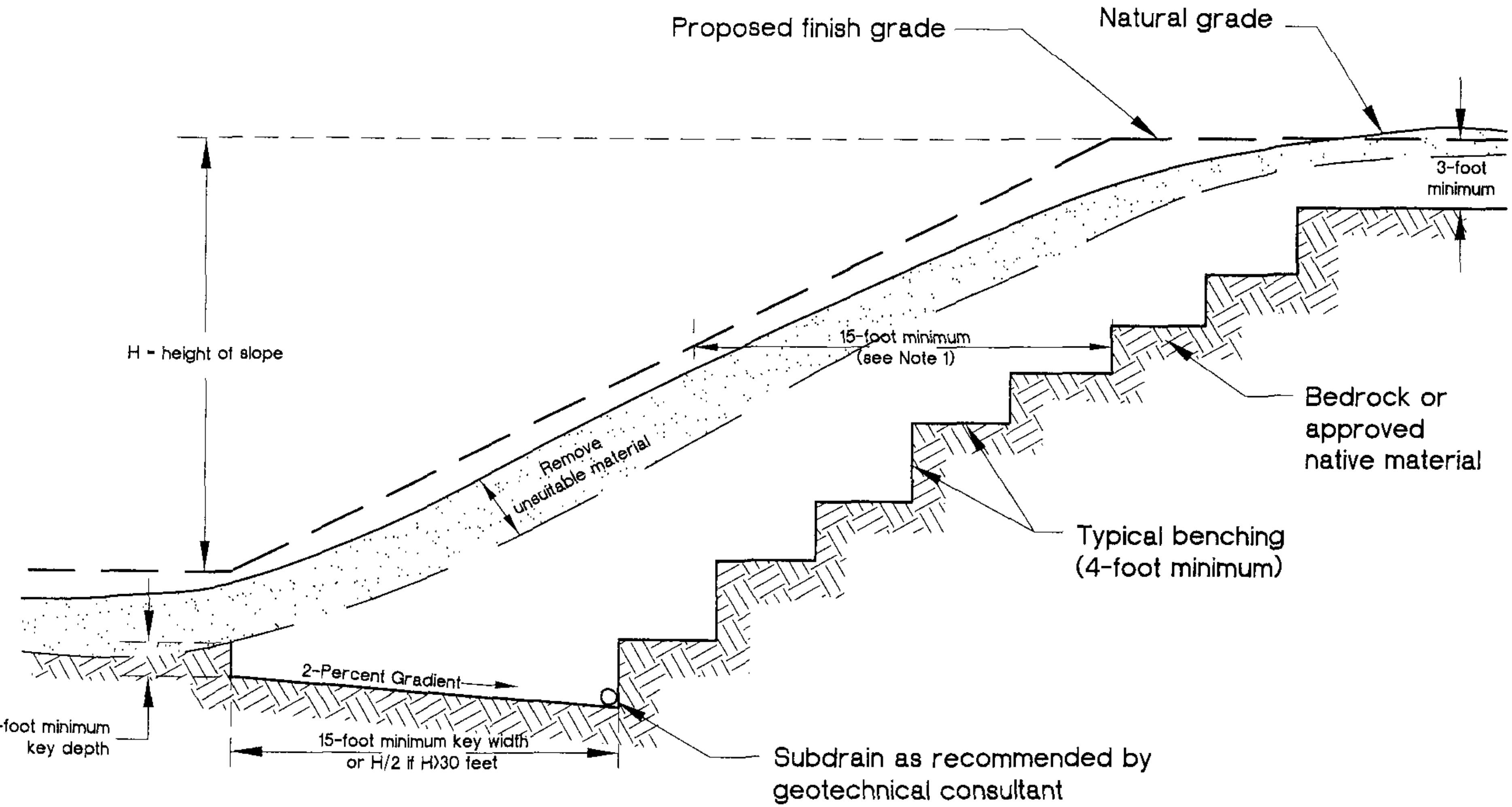


NOTES:

1. Subdrains may be required as specified by the geotechnical consultant.

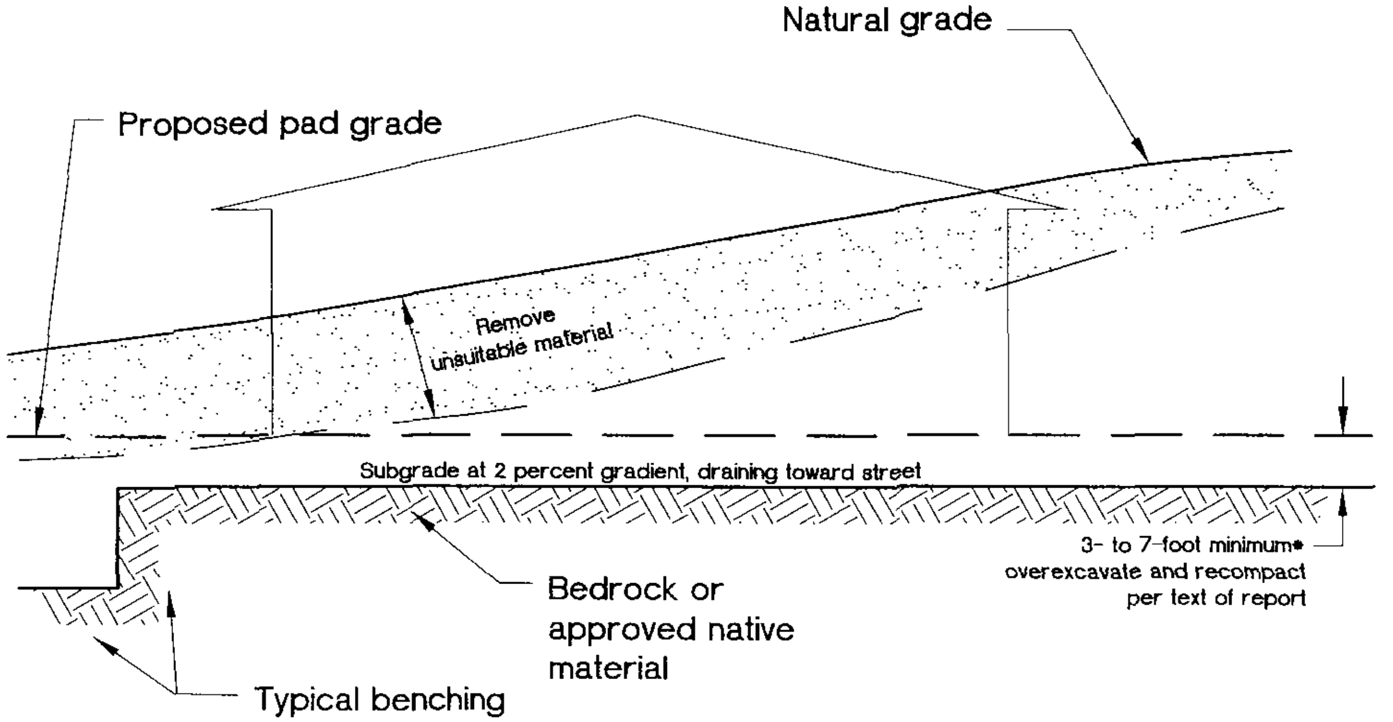
2. W shall be equipment width (15 feet) for slope heights less than 25 feet. For slopes greater than 25 feet, W shall be evaluated by the geotechnical consultant. At no time, shall W be less than H/2, where H is the height of the slope.

GeoSoils, Inc.	STABLIZATION FILL FOR UNSTABLE MATERIAL EXPOSED IN CUT SLOPE DETAIL	Plate E-9

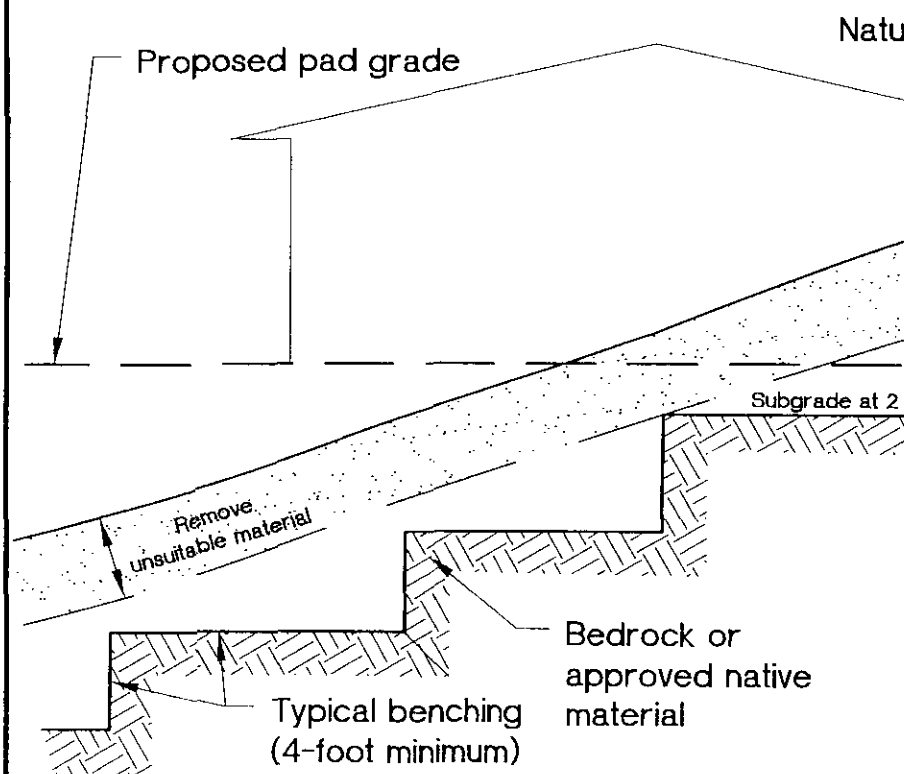


NOTES:
1. 15-foot minimum to be maintained from proposed finish slope face to backcut.
2. The need and disposition of drains will be evaluated by the geotechnical consultant based on field conditions.
3. Pad overexcavation and recompaction should be performed if evaluated to be necessary by the geotechnical consultant.

GeoSoils, Inc.	SKIN FILL OF NATURAL GROUND DETAIL	Plate E-10



CUT LOT OR MATERIAL-TYPE TRANSITION

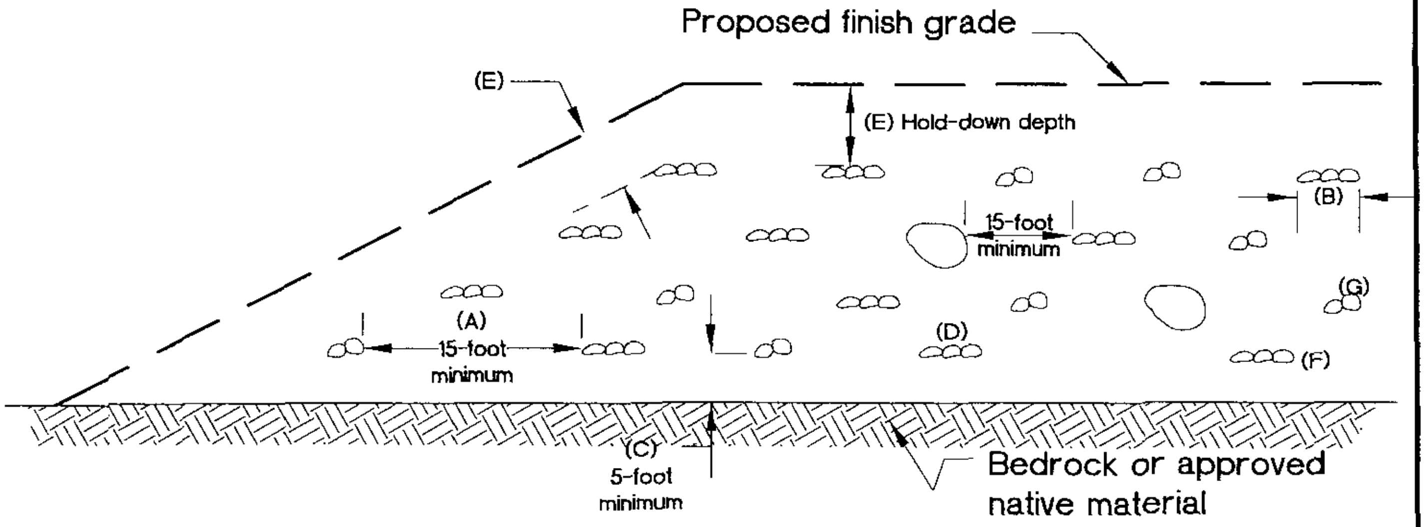


* Deeper overexcavation may be recommended by the geotechnical consultant in steep cut-fill transition areas, such that the underlying topography is no steeper than 3:1 (H:V)

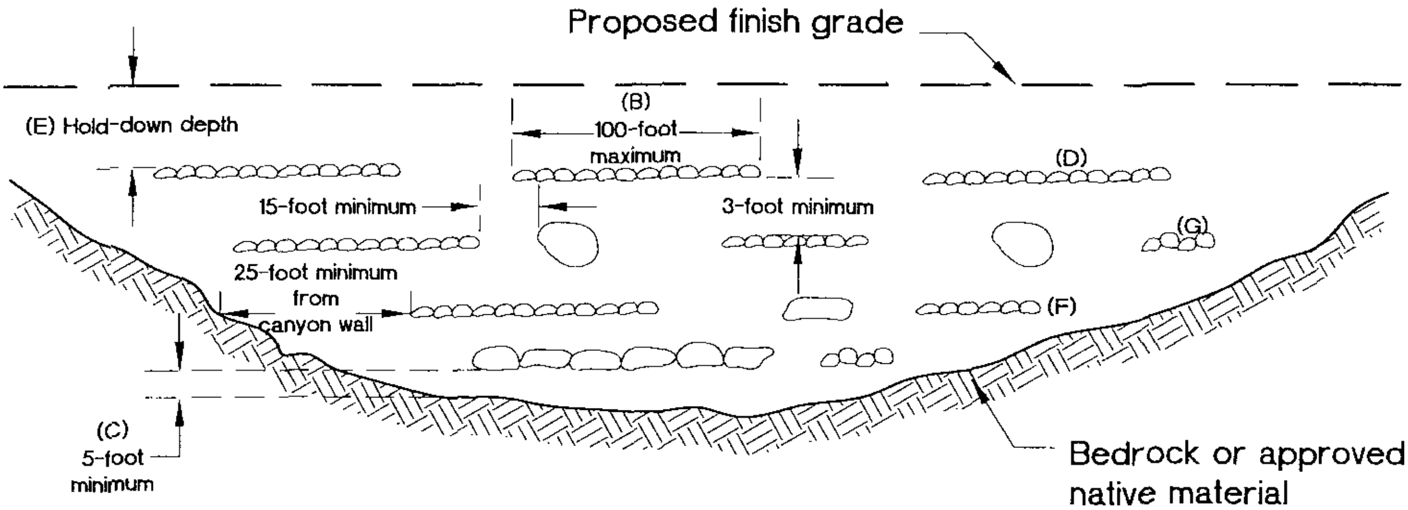
CUT-FILL LOT (DAYLIGHT TRANSITION)

GeoSoils, Inc.	TRANSITION LOT DETAILS	Plate E-12

VIEW NORMAL TO SLOPE FACE



VIEW PARALLEL TO SLOPE FACE



NOTES:

A. One equipment width or a minimum of 15 feet between rows (or windrows).

B. Height and width may vary depending on rock slze and type of equipment. Length of windrow shall be no greater than 100 feet.

C. If approved by the geotechnical consultant, windrows may be placed direcity on competent material or bedrock, provided adequate space is available for compaction.

D. Orientation of windrows may vary but should be as recommended by the geotechnical engineer and/or engineering geologist. Staggering of windrows is not necessary unless recommended.

E. Clear area for utility trenches, foundations, and swimming pools; Hold-down depth as specified in text of report, subject to governing agency approval.

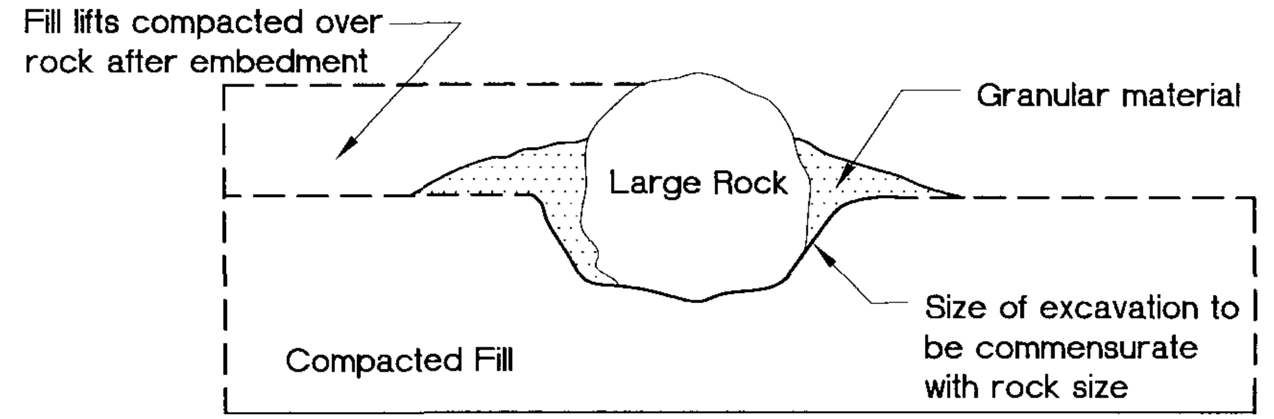
F. All fill over and around rock windrow shall be compacted to at least 90 percent relative compaction or as recommended.

G. After fill between windrows is placed and compacted, with the lift of fill covering windrow, windrow should be proof rolled with a D-9 dozer or equivalent.

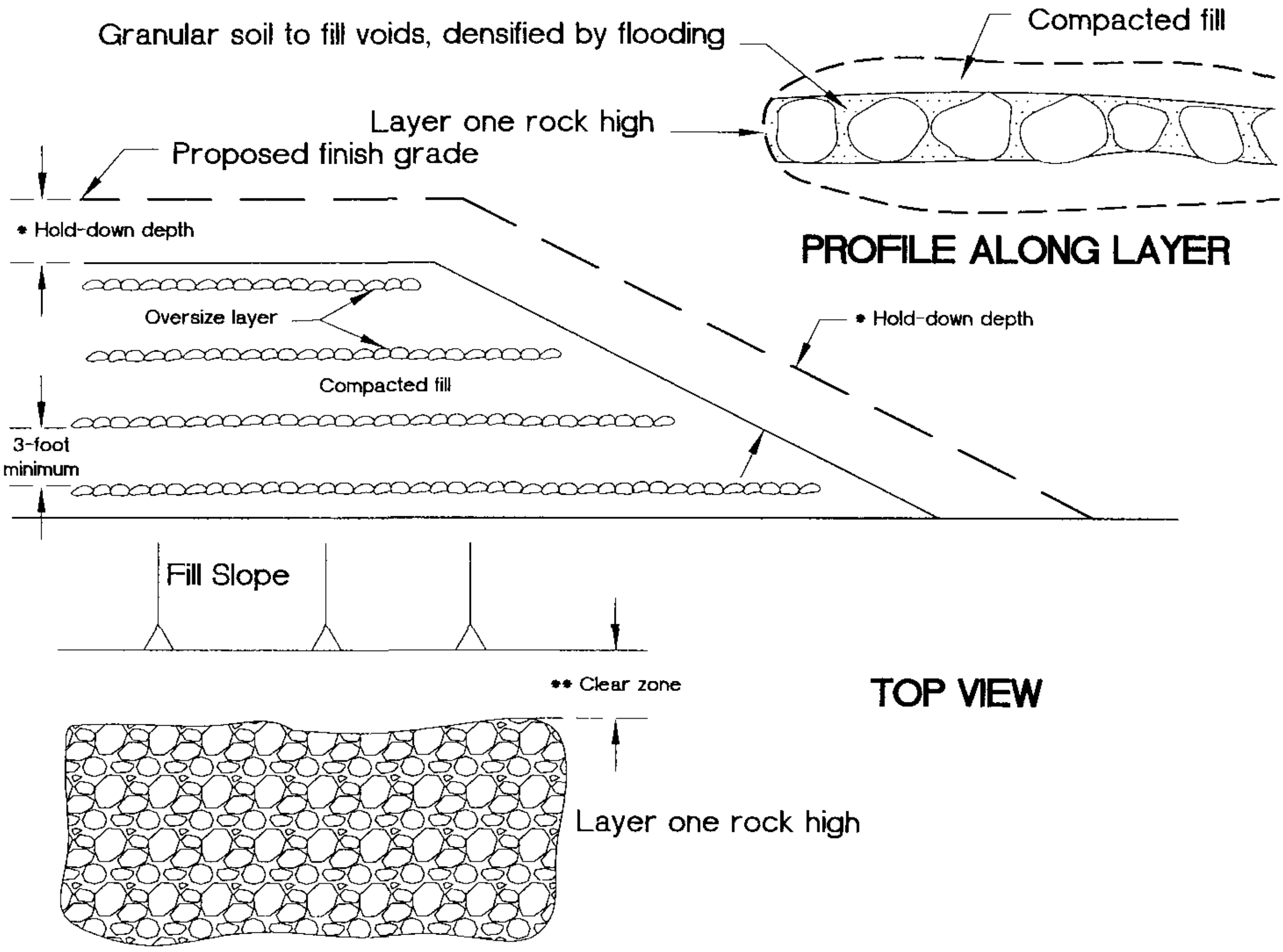
VIEWS ARE DIAGRAMMATIC ONLY AND MAY BE SUPERSEDED BY REPORT RECOMMENDATIONS OR CODE
ROCK SHOULD NOT TOUCH AND VOIDS SHOULD BE COMPLETELY FILLED

GeoSoils, Inc.	OVERSIZE ROCK DISPOSAL DETAIL	Plate E-13

ROCK DISPOSAL PITS



ROCK DISPOSAL LAYERS



* Hold-down depth or below lowest utility as specified in text of report, subject to governing agency approval.

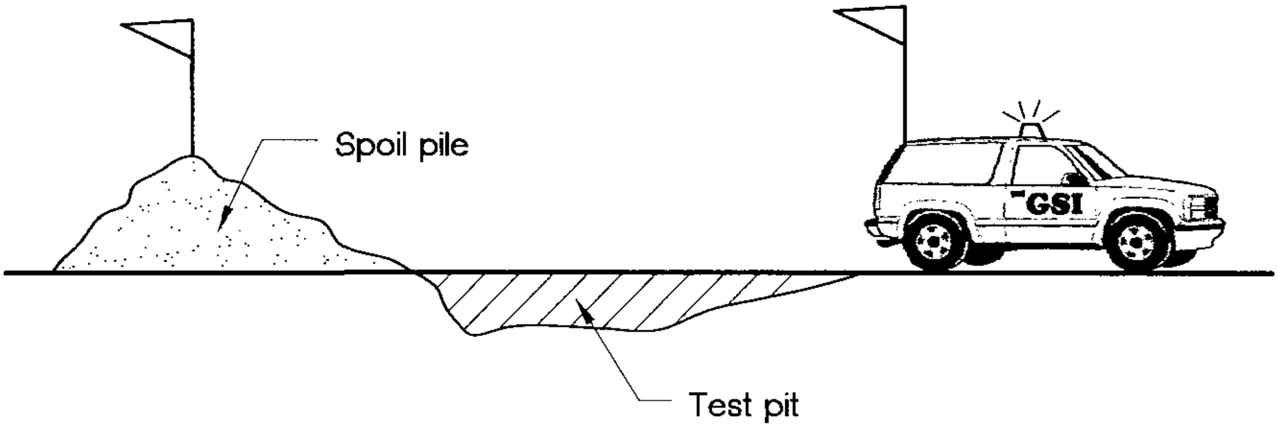
** Clear zone for utility trenches, foundations, and swimming pools, as specified in text of report.

VIEWS ARE DIAGRAMMATIC ONLY AND MAY BE SUPERSEDED BY REPORT RECOMMENDATIONS OR CODE

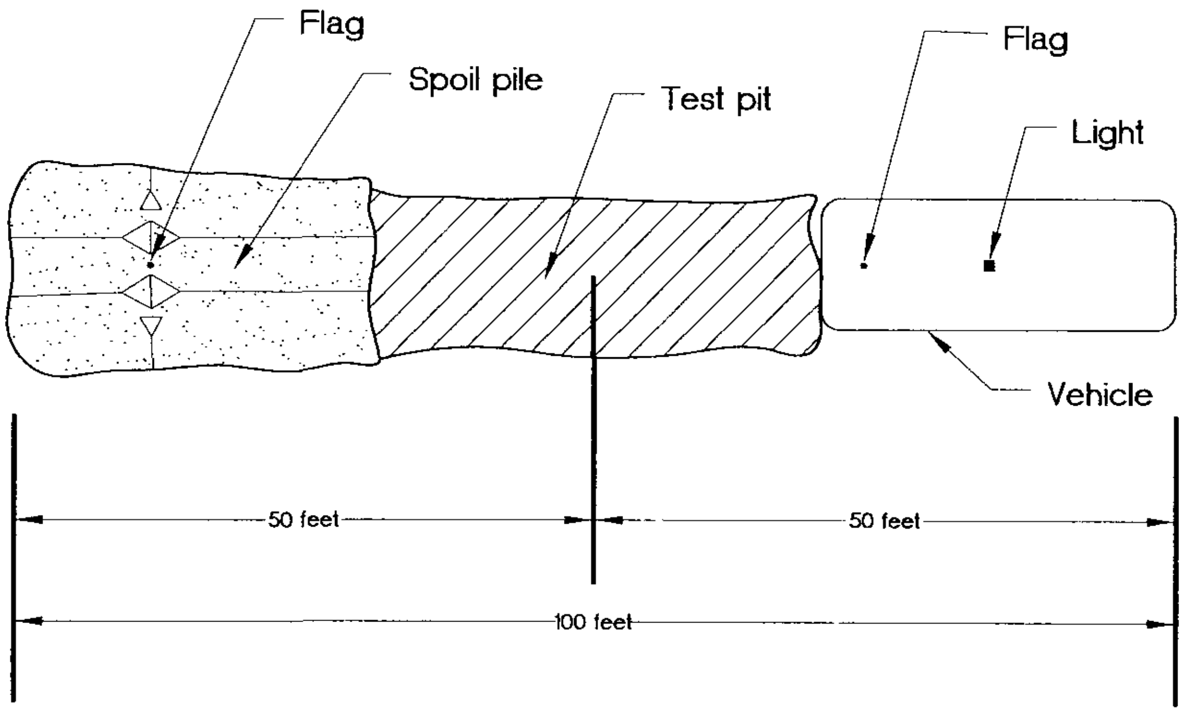
ROCK SHOULD NOT TOUCH AND VOIDS SHOULD BE COMPLETELY FILLED IN

GeoSoils, Inc.	ROCK DISPOSAL DETAIL	Plate E-14

SIDE VIEW



TOP VIEW



GeoSoils, Inc.

RIVERSIDE CO.
ORANGE CO.
SAN DIEGO CO.

GEOTECHNICAL MAP

Plate 1 of 1

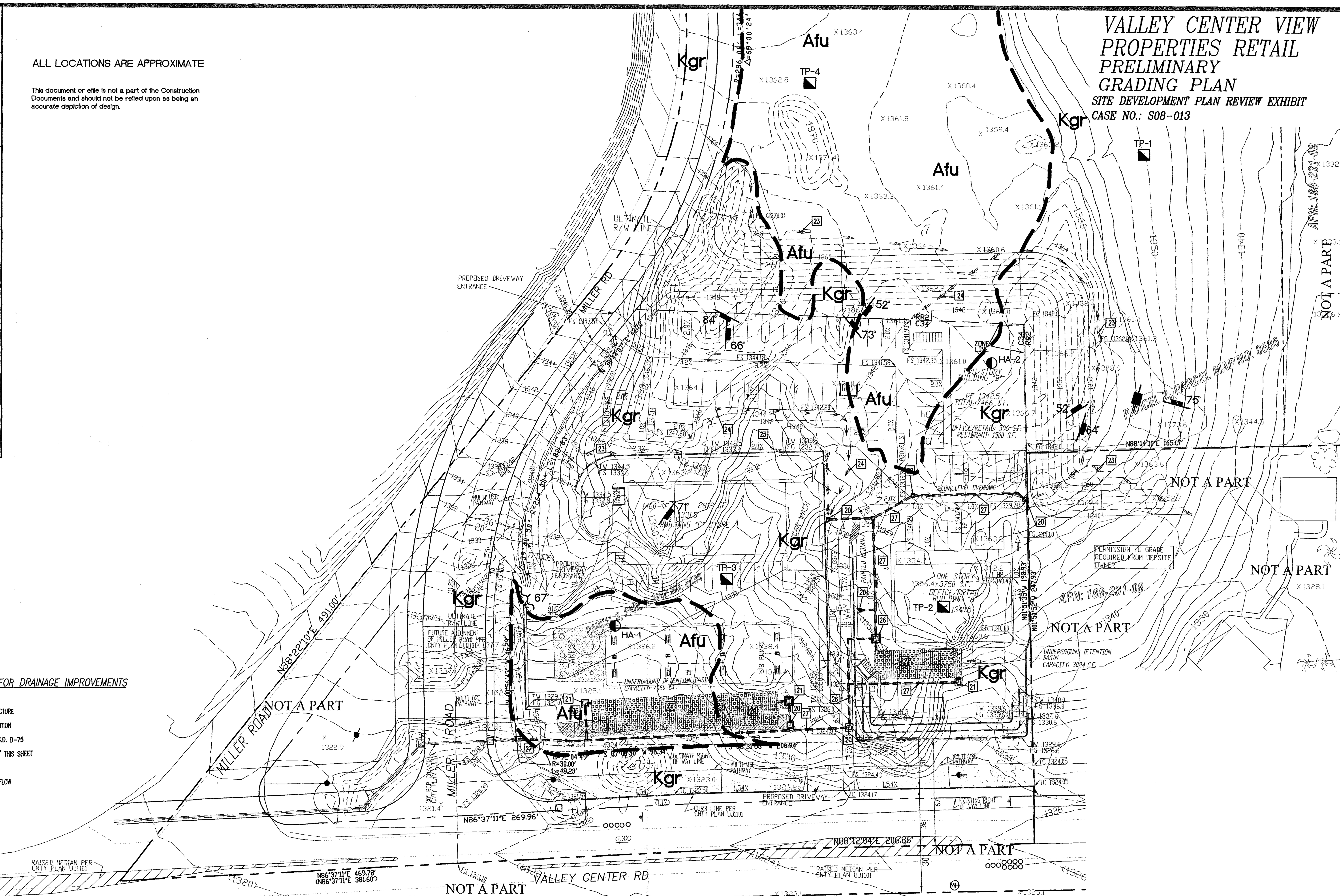
W.O. 5654-A2-SC	DATE 02/09	SCALE 1"=30'

LEGEND

Afu	Artificial fill - undocumented
Kgr	Cretaceous granitics
	Approximate location of geologic contact
30°	Attitude of inclined shear
71°	Attitude of inclined joint
	Attitude of vertical joint
HA-2	Approximate location of exploratory hand-auger boring
TP-12	Approximate location of exploratory test pit

ALL LOCATIONS ARE APPROXIMATE

This document or efile is not a part of the Construction Documents and should not be relied upon as being an accurate depiction of design.



CONSTRUCTION LEGEND FOR DRAINAGE IMPROVEMENTS

20	CATCH BASIN
21	DETENTION BASIN MONITORING STRUCTURE
22	UNDERGROUND STORM WATER DETENTION
23	CONCRETE BROW DITCH PER S.D.R.S.D. D-75
24	GRASS LINED SWALE PER DETAIL "A" THIS SHEET
25	OPEN CELL GRASS PAVEMENT
26	DETENTION BASIN EMERGENCY OVERFLOW
27	STORM DRAINAGE PIPE

PROPERTY OWNER INFORMATION

NAME: VALLEY CENTER VIEW PROPERTIES
ADDRESS: 3936 HORTENSIA STREET
SAN DIEGO, CA 92110
TELEPHONE NUMBER: 619-523-0133
(24 HOUR CONTACT NUMBER)
SITE A.P.N. NUMBER: 188-231-34
SITE ADDRESS: xxxxx VALLEY CENTER ROAD

30 0 30 60
Scale: 1" = 30'

CIVIL ENGINEERING • LAND PLANNING
Aquaterra Engineering Inc.
1843 Campesino Place
Oceanside, CA 92054
Tele: (760) 439-2802
Fax: (760) 439-2866

SCALE: 1"= 30'

PRELIMINARY DRAINAGE PLAN

PLAN CHECK/PERMITS

BUILDING PERMIT
PLAN CHECK NUMBER: ____

PARCEL MAP NUMBER: ____

ENGINEER OF WORK

I HEREBY DECLARE THAT I AM THE ENGINEER OF WORK FOR THIS PROJECT AND THAT I HAVE EXERCISED RESPONSIBLE CHARGE OVER THE DESIGN OF THE PROJECT.

NAME: GARY LIPSKA DATE: ____
RCE NO: RCE 23080 EXPIRES: 12/31/09

PRIVATE CONTRACT

COUNTY OF SAN DIEGO
DEPARTMENT OF PLANNING AND LAND USE

PRELIMINARY GRADING PLAN FOR:
VALLEY CENTER VIEW
PROPERTIES RETAIL

SHEET: 3 OF SHEETS: 3

APPROVED
DIRECTOR OF PLANNING AND LAND USE
BY: DATE:

GRADING PERMIT NUMBER:

plot date: 12-15-08