PRELIMINARY GEOTECHNICAL INVESTIGATION
COMMERCIAL AND RESIDENTIAL DEVELOPMENT
±4.87-ACRE PARCEL, APN 381-030-005
LAKE ELSINORE, RIVERSIDE COUNTY, CALIFORNIA

FOR

MR. RON JIRON
601-C CRANE STREET
LAKE ELSINORE, CALIFORNIA 92530

W.O. 5043-A-SC MAY 25, 2006

GeoSoils, Inc.

Geotechnical • Coastal • Geologic • Environmental

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May 25, 2006

W.O. 5043-A-SC

Mr. Ron Jiron
601-C Crane Street
Lake Elsinore, California 92530

Subject: Preliminary Geotechnical Investigation, Commercial and Residential Development, ±4.87-Acre Parcel, APN 381-030-005, Lake Elsinore, Riverside County, California

Dear Mr. Jiron:

In accordance with your request and authorization, this report presents the results of GeoSoils, Inc.'s (GSI) preliminary geotechnical investigation of the subject property. The main purpose of this study was to evaluate the existing geotechnical conditions relative to the proposed development. A secondary purpose was to evaluate remedial removal depths, and provide preliminary foundation design criteria, as well as earthwork guidelines for the site.

EXECUTIVE SUMMARY

Based on our review of data (see Appendix A), field exploration, laboratory testing, and geologic and engineering analyses, the proposed site appears suitable for its intended use as a mixed commercial and residential development, from a geotechnical viewpoint, provided the recommendations presented in the text of this report are implemented during planning, design, and construction of the project.

- Geotechnical developmental considerations include thick potentially compressible alluvium, and site seismicity and secondary seismic concerns.

- Regional seismic shaking, ranging from moderate to severe, may occur on the site associated with nearby and/or regional faults. Accordingly, the proposed structures and foundations should be designed to resist seismic forces in accordance with the criteria contained in the Uniform Building Code/California Building Code (International Conference of Building Officials [ICBO], 1997 and 2001) for Seismic Zone 4, and the seismic parameters provided herein.

- The site <u>does</u> lie within a liquefaction zone established by the County of Riverside.

- Based on laboratory and field observations, the soil materials encountered throughout the site are expected to generally be very low in expansion potential; however, the presence of low to medium expansive soils can not be precluded. Based on preliminary test results, the soils generally have negligible sulfate contents per the UBC/CBC (ICBO, 1997 and 2001). Accordingly, the use of sulfate resistant concrete is not anticipated, based on the available data. Due to the proximity of this project to the lake, corrosion protection for buried metallic improvements may be necessary. Accordingly, a consulting corrosion engineer should be retained to provide specific recommendations for foundations, piping, etc.

- Subsurface water is generally not anticipated to adversely affect site development, provided that the recommendations contained in this report are properly incorporated into final design and construction. These observations reflect site conditions at the time of our investigation and do not preclude future changes in local groundwater conditions from excessive irrigation, precipitation, or that were not obvious, at the time of our investigation. The potential for perched groundwater conditions, both during grading, as well as after site development, may not be precluded. This potential should be disclosed to all interested parties, including homeowners and any homeowners association. Dewatering may be necessary during remedial removals in the northeastern portion of the site at the location of the proposed pool. High groundwater indicates increased slab moisture protection for slab-on-grade floors.

- Remedial removals across the site are generally anticipated to be on the order of about ±4 to ±7 feet with the potential for locally deeper removals (due to dry or porous materials), up to about ±15 feet. Local remedial grading to in the area of the existing pool and remnant foundations will be necessary and may exceed these estimates. The field testing criteria for the base of remedial removals at the site is provided herein. The removal bottoms should be greater than or equal to about 85 percent saturation, and/or greater than or equal to about 105 pounds per cubic foot (pcf) dry density for in-place native materials, which has been acceptable in various agency jurisdictions throughout various counties. The exposed removal surface should be scarified to a depth of 12 inches, moisture conditioned (if necessary), and then compacted prior to fill placement to finish pad grade. Again, localized deeper removals may be necessary due to buried drainage channel meanders, or dry, porous materials, up to about ±15 feet deep. The project soils engineer/geologist should observe and evaluate all removal areas during the grading.

- On a preliminary basis, removals at the property boundaries will likely be restricted unless permission is granted by adjacent property owners to perform offsite remedial removals. Based on the anticipated removal depths, such offsite removals, for structural areas, would likely be approximately ±5 feet of encroachment, for a 5-foot removal depth, into the adjacent properties. The scenarios below assume no offsite settlement-sensitive structures exist within 5 feet of the property line.

Assuming offsite earthwork is not performed, onsite removals may consist of removal of ±5 feet in depth on a 1:1 projection from the property line into the project. On a preliminary basis, a structural setback of approximately ±10 feet, from the property lines would then be required. In lieu of setbacks, more conservative post-tension foundation or deep foundation (e.g., caissons) designs may be utilized for any site improvements. Design parameters for such foundation systems can be provided upon request. Depending upon actual removal depth and extent attained during grading, revised setbacks may be offered. Any settlement sensitive improvements constructed within the zone of limited or no removals may be subject to settlement and distress. This potential will need to be disclosed to all homeowners and any homeowners association.

- Due to the complexity of the pool/confining sheet pile wall near the lakeshore, GSI recommends that the pool structure be pile supported. In addition, GSI recommends that an under-pool leak detection blanket also be provided.

- Based on available data, blasting does not appear to be necessary to achieve design grades. This does not preclude difficult excavation of utilities near the southwestern portion of the site.

- Based on our experience on similar projects, proposed fill and cut slopes constructed using onsite materials should be grossly and surficially stable provided the recommendations contained herein are implemented during site development. Significant cut slopes, exposing poorly consolidated and/or sandy alluvial materials would likely require stabilization and/or erosion protection.

- Due to the high level of anticipated ground shaking, GSI recommends that post-tension foundation systems be used for the proposed buildings.

- Our review indicates no known active faults are specifically crossing the site area, and the site is <u>not</u> within an Alquist-Priolo Earthquake Fault Zone (Hart and Bryant, 1997, or a known County fault zone). Further, aerial photo-lineaments (typically associated with active faulting), do not transect the site.

- Secondary seismic hazards include a seiche (standing wave oscillation) within the lake during or shortly after an earthquake. The approximate height of the seiche will be approximately the same as the design boat wake (see boat wake section) and is already mitigated through typical shoreline protection.

- It is our understanding that an environmental site assessment of this site was performed by others under a separate cover. This report is not intended to be used in lieu of an environmental site assessment.

- Adverse geologic features that would preclude project feasibility were not encountered.

- Based on our review of site conditions, complete excavation and recompaction of any liquefiable soils, 20 to 40 feet below the ground surface, does not appear to be geotechnical feasible, based on the available data and, given the water table and the limited land available for earthwork of this magnitude.

- 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 our office.

Respectfully submitted,

GeoSoils, Inc.

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SCOPE OF SERVICES

The scope of our services included the following:

1. Review of readily-available geologic data for the area (see Appendix A) including stereoscopic aerial photographs.

2. Subsurface exploration, consisting of six (6) hollow stem auger borings, as well as 11 Cone Penetrometer Test (CPT) soundings, was performed to evaluate the near surface and deeper soil and geologic and hydrogeologic conditions (see Appendix B).

3. General site seismicity (Appendix C).

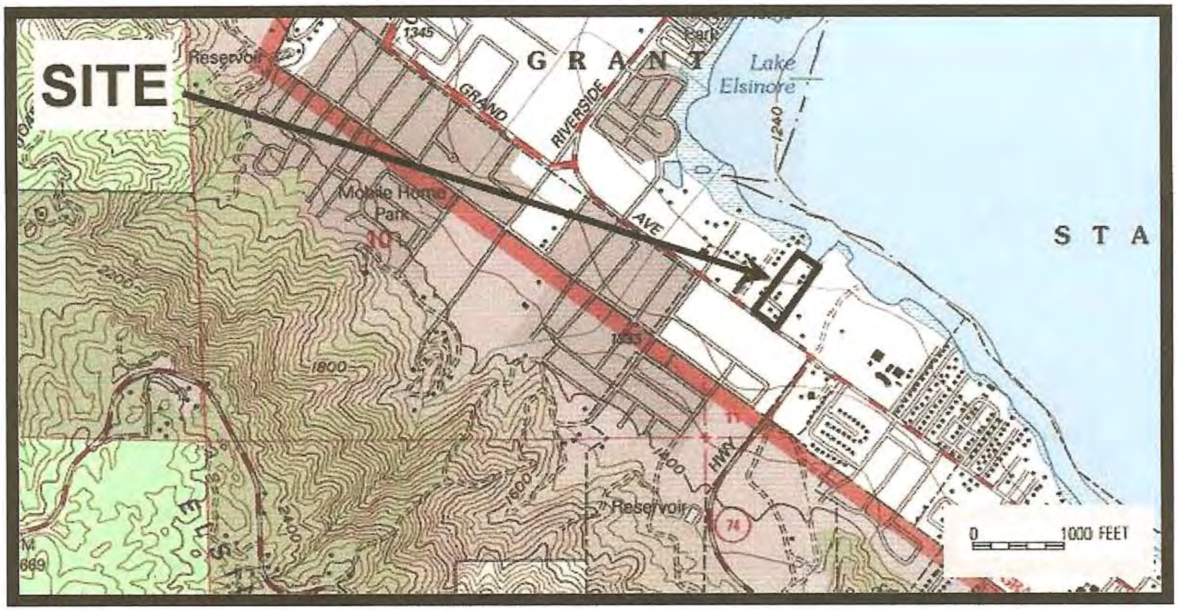
4. Pertinent laboratory testing of representative soil samples collected during our subsurface exploration program (Appendix D).

5. Appropriated geologic and engineering analysis of the data collected (Appendix E).

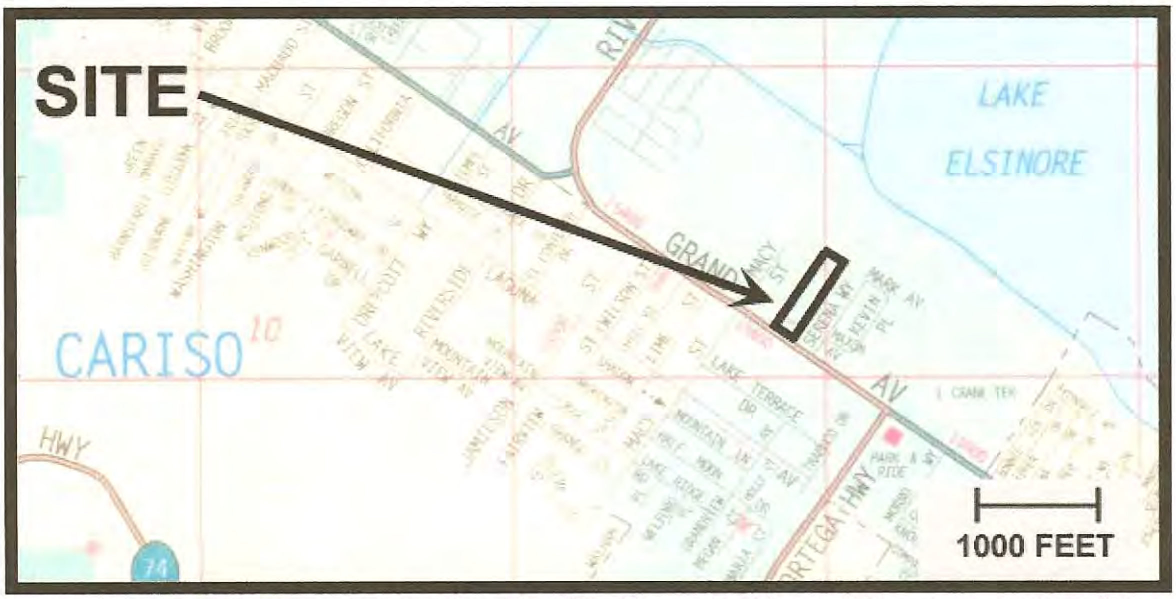
6. Preparation of this report presenting our conclusions and recommendations with respect to geotechnical design criteria and considerations, including boring logs, laboratory test results, regional seismic data, general earthwork factors, recommendations for site grading, and preliminary foundation design.

SITE DESCRIPTION

The project area consists of roughly ±4.87 acres of relatively gently sloping terrain located in the City of Lake Elsinore, Riverside County, California (see Figure 1, Site Location Map). The rectangular-shaped property consists of an old motel site along Grand Avenue. It is likely that some grading/earthwork was performed in the past to create the building pad(s) for the motel structure(s). However, the nature of any such old earthwork is not readily apparent as the site has been demolished and the remaining "improvements" generally consist of old concrete pads, an empty swimming pool, an old boat dock, and a sewer pump station. It was not readily apparent from our field investigation to what the extent previous and/or abandoned utilities remain onsite following the previous demolition efforts. Most likely some of these utilities from earlier development will be encountered during grading of the site. A Riverside County Flood Control District (RCFCD) drainage channel is located along the southeastern border of the property (see Plate 1, Geotechnical Map).



Base Map: TOPO!® ©2003 National Geographic, U.S.G.S. Lake Elsinore and Alberhill Quadrangles, California–Riverside Co., 7.5-Minute, dated 1997.



Base Map: The Thomas Guide, Riverside Co. Street Guide and Directory, 2005 Edition, by Thomas Bros. Maps, page 865.

LOCATION AND SCALES APPROXIMATE

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SITE LOCATION MAP	
Figure 1	

Elevations within the site vary between approximately 1,247 feet Mean Sea Level (MSL) to 1,290 feet MSL, thus overall topographic relief is on the order of ±43 feet. Relatively minor amounts of undocumented fill material was noted in several locations across the property.

However, one area of fill was noted at the northeastern end of the property, likely associated with prior grading to create a relatively level pad area near the margin of Lake Elsinore. Based upon a limited review of background data in our files and site reconnaissance, it appears that the property is underlain by an unknown quantity and depth of undocumented fill, as well as young alluvial valley and/or fan deposits. It is also likely that granitic bedrock underlies the site at an unknown depth. While the site does not lie within a State of California, Alquist Priolo earthquake fault zone, it does lie within a County of Riverside hazard zone for liquefaction. Based on our review of published maps, as well as County fault hazard zone mapping for the Willard fault, the site appears to be located between mapped traces and hazard zones of this fault, and does not appear to lie within such a County fault zone. The southwestern trace of the fault has been mapped approximately ±490 feet southwest of the site and the northeastern trace appears to lie some ±945 feet northeast of the site.

PROPOSED DEVELOPMENT

Based on our conversations and review of site development plans by ADKVAS Group (ADKVAS, 2005), GSI understands that proposed development of the site will consist of construction of a commercial structure, as well as a nine-building, two- or three-story condominium development with associated parking and utility improvements. In addition, a swimming pool is proposed in the northeastern portion of the site, along the edge of Lake Elsinore. This pool area is shown on the current plans to be constructed in conjunction with a proposed retaining/crib wall, which will allow vehicle access to the boat launch area. The existing dock will remain and be improved as part of the project.

It is our understanding that conventional cut and fill grading techniques would be necessary to bring the site to design grades for the development. We further understand that buildings are proposed as two- and/or three-story structures, with slab-on-grade/continuous and/or deepened footings, utilizing wood-frame type construction. Building loads are assumed to be typical for this type of relatively light construction. Sewage disposal is anticipated to be accommodated by tying into the regional system. A new sewer pump station near the northeastern portion of the site will be necessary for the onsite sanitary system. It is our understanding that importing of soils for development of the site will not be allowed by the City; however, this would likely exclude the importation of road base and/or utility bedding.

FIELD STUDIES

Field studies conducted during the our investigation of the site consisted of the following:

1. Geologic reconnaissance and mapping.

2. Excavation of six hollow stem auger borings as well as 11 CPT soundings to evaluate the near surface and deeper soil and geologic and hydrogeologic conditions (see Appendix B). The borings were logged by a registered professional geologist from our firm who collected representative bulk and undisturbed samples for appropriate laboratory testing from the borings. Logs of the borings and CPT results are presented in Appendix B. Groundwater encountered during our borings and CPT soundings was noted and included pore pressure measurements. The approximate locations of the borings and CPT soundings are presented on Plate 1 (Geotechnical Map).

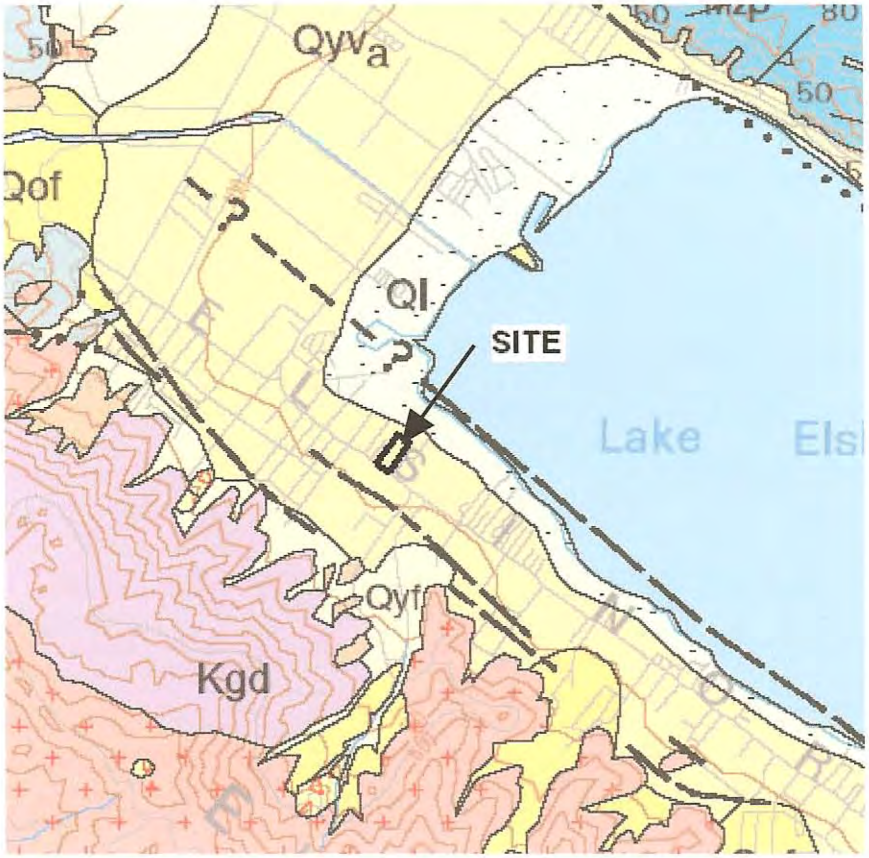
GEOLOGY

Regional Geologic Setting

The site is located on the western margin of the Perris Block, a portion of a prominent natural geomorphic province in southwestern California known as the Peninsular Range. The Peninsular Range is characterized by steep, elongated ranges and valleys that trend northwesterly. The Santa Ana Mountains lie along the western side of the Elsinore fault zone, and the Perris Block is located along the eastern side of the fault zone. This province is typified by plutonic and metamorphic rocks (bedrock) which comprise the majority of the mountain masses, with relatively thin volcanic and sedimentary deposits discontinuously overlying the bedrock, and with alluvial fan deposits filling in the valleys, and younger alluvium filling in the incised drainages. The alluvial deposits are derived from the water borne deposition of the products of weathering and erosion of the bedrock.

Local Geology and Site Earth Materials

As mapped by Weber (1977), the site was characterized as generally being underlain by late Pleistocene-age flood plain and valley fill deposits. Holocene lacustrine (lake) deposits were locally mapped along the shoreline area of Lake Elsinore. Mapping by Morton and Weber (2003) indicate the site to be underlain by Holocene and late Pleistocene alluvial fan deposits. Similarly, Holocene lacustrine (lake) deposits were noted along the shoreline area. Subsequent mapping by Morton (2004) continued to map the site as being underlain by the same materials (See Figure 2, Local Site Geologic Map).



Principal Site Geologic Unit

Qyv - Young alluvial valley deposits (Holocene and Late Pleistocene)

SCALE

0 1 2

MILES

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LOCAL SITE GEOLOGIC MAP

- from Morton (2004)

Figure 2

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For the purposes of this report, the geologic units observed (during this study) within the project site generally consist of undocumented artificial fill materials, colluvium, and late Pleistocene-age alluvial fan deposits. Map symbols refer to the near surface sediments and geologic units shown on Plate 1. These units are described as follows, from youngest to oldest.

Artificial Fill - Undocumented (Map Symbol - Afu)

An area of artificial fill was clearly noted in the northeastern most portion of the property. This fill was likely placed during previous onsite grading to create a relatively level pad area near the margin of Lake Elsinore. This fill was encountered in Boring B-1 and CPT-1, during our site exploration. The fill consisted of grayish brown to dark brown silty sand, which was dry to moist and loose to medium dense. Other areas within the property are also likely underlain by relatively minor amounts of undocumented fill; however, such areas were not readily apparent based on reconnaissance mapping and were not encountered in our site explorations. As a result of the potentially compressible nature of these soils, they are considered unsuitable for support of structures and/or improvements in their existing state. These materials should be removed, moisture conditioned, and recompacted, in areas where settlement-sensitive improvements are proposed.

Colluvium (Not Mapped)

Minor colluvial soils were also locally present within the site. Such materials are likely intermixed with minor artificial fills and are therefore, undifferentiated and unmapped herein. Due to the potentially compressible nature of such surficial soils, they are considered unsuitable for support of structures and/or improvements in their existing state. Therefore, these soils will be need to be removed, moisture conditioned, and recompacted, if encountered during planned excavation, in areas to receive settlement-sensitive improvements, or within their influence.

Lacustrine Deposits (Not Mapped)

Holocene lacustrine (lake) deposits have been mapped offsite along the margin of Lake Elsinore by Weber (1977), Morton and Weber (2003), and Morton (2004). Based on the mapping by these authors, as well as GSI's field observations and investigations, it is likely that the site is not underlain by these deposits. No such materials were encountered in our site mapping or explorations. Such materials are likely located offsite beneath the current level of Lake Elsinore. Although unlikely, should such materials be encountered during any future site earthwork, they would be considered unsuitable for support of structures and/or improvements in their existing state. Therefore, these soils will be need to be removed, moisture conditioned, and recompacted, if encountered during planned excavation, in areas to receive settlement-sensitive improvements, or within their influence.

Pleistocene Alluvial Fan Deposits (Map Symbol - Qf)

Pleistocene alluvial fan deposits (Morton and Weber, 2003; and Morton, 2004) are the predominant earth material noted during our field mapping and site explorations, although Morton (2004) has indicated possibly early Holocene and late Pleistocene-age for these materials. These sediments were generally observed to be mostly brown to dark brown to grayish brown and reddish brown, silty sand, clayey sand, with minor sand and clayey sand interbeds. Consistency was generally medium dense to dense where sandy and medium stiff to hard where silty or clayey. Moisture content within the near surface areas of the fan deposits differed from dry to damp. At depth, moisture was generally moist and wet to saturated. Based on the available data, the upper weathered portions (i.e., from ±5 to ±10 feet) of these deposits are potentially compressible and collapsible, and considered unsuitable for support of structures and/or improvements in their existing state. Therefore, these near-surface soils will be need to be removed, moisture conditioned, and recompacted, if encountered during planned excavation, in areas to receive settlement-sensitive improvements, or within their influence.

FAULTING AND REGIONAL SEISMICITY

The site is situated in an area of active as well as potentially-active faults. The Elsinore fault zone (located approximately ±3 miles northeast of the site) is considered active and is included within an Alquist-Priolo Earthquake Fault Zone. Our review indicates that there are no known active faults crossing the site, and the site is not within a Fault-Rupture Hazard Zone (Hart and Bryant, 1997). Based on our review of published maps, as well as County fault hazard zone mapping for the Willard fault, the site appears to be located between mapped traces and hazard zones associated with this fault. The southwestern trace has been mapped approximately ±490 feet southwest of the site and the northeastern trace appears to lie some ±945 feet northeast of the site (see Figure 2).

Photolineament Analysis

Due to the proximity of mapped traces of the Willard fault, and in order to evaluate if any fault-related features could be identified, a lineament analysis was performed using stereoscopic "false-color" infrared aerial photographs (see Appendix A) at a scale of 1:40,000. No lineaments were observed transecting the site.

Seismicity

The possibility of ground shaking at the site may be considered similar to the southern California region as a whole. The acceleration-attenuation relations of Campbell and Bozorgnia (1994 and 1997), and Bozorgnia, and others (1999) have been incorporated into EQFAULT (Blake, 2000a). For this study, peak horizontal ground accelerations anticipated at the site were determined based on the mean and mean plus 1 - sigma attenuation curves developed by those investigators. These acceleration-attenuation relations have been incorporated into EQFAULT (Blake, 2000a), a computer program which performs deterministic seismic hazard analyses using 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 user-selected acceleration-attenuation relations that are contained in EQFAULT. Based on the EQFAULT program, peak horizontal ground accelerations (deterministic acceleration values) from an upper bound event at the site may be on the order of 0.51g to 0.83g.

Historical site seismicity was evaluated with the acceleration-attenuation relations of Bozorgnia, and others (1999) and the computer program EQSEARCH (Blake, 2000b). This program was utilized to perform a search of historical earthquake records for magnitude 5.0 to 9.0 seismic events within a 100-mile radius, between the years 1800 to June, 2005. Based on the selected acceleration-attenuation relation, a peak horizontal ground acceleration has been estimated, which may have affected the site during the specific seismic events in the past. Based on the available data and attenuation relationship used, the estimated maximum (peak) site acceleration during the period 1800 to June, 2005, was 0.52g. In addition, a seismic recurrence curve is also estimated/generated from the historical data (see Appendix C).

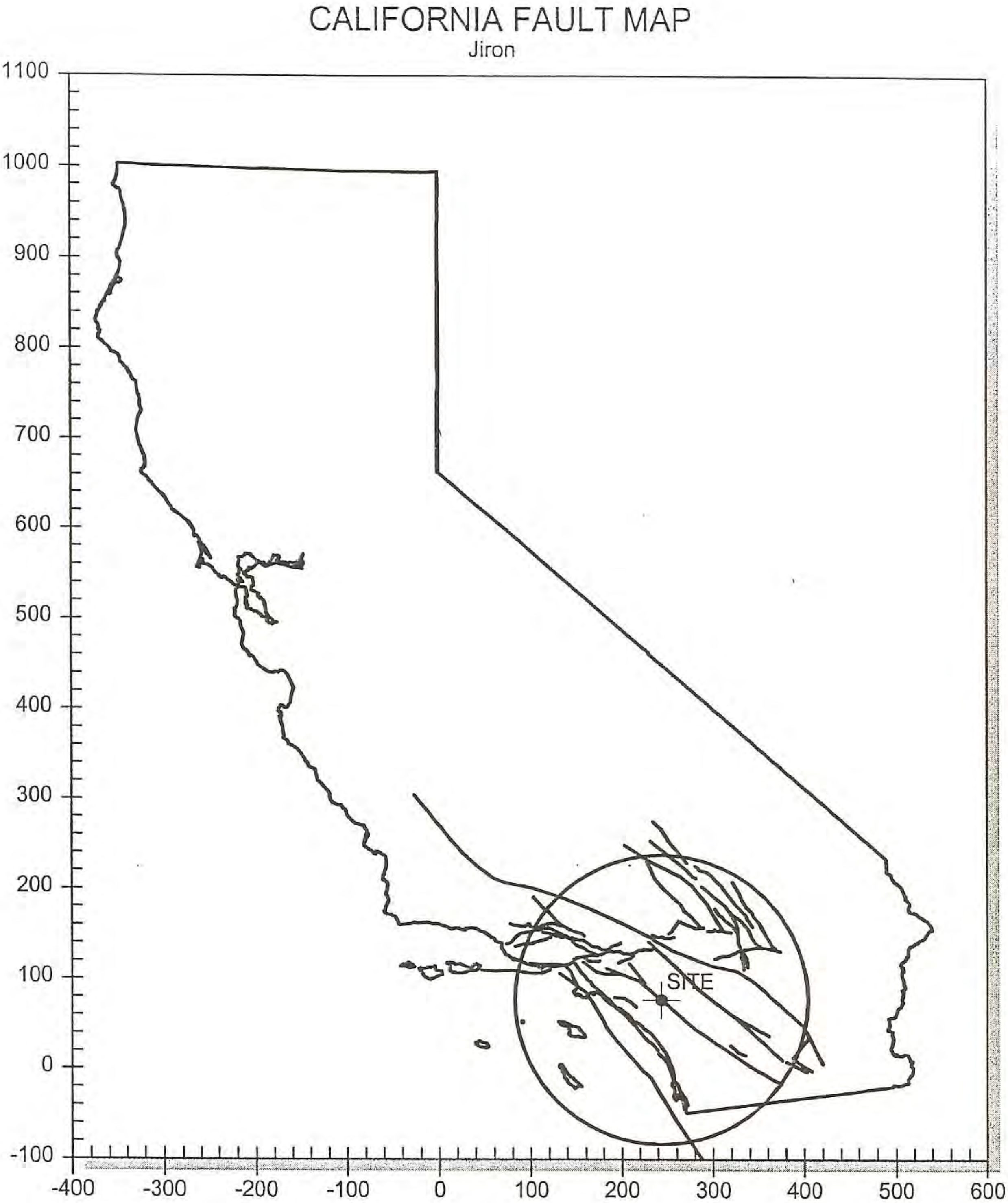
A probabilistic seismic hazards analyses was performed using FRISKSP (Blake, 2000c) which models earthquake sources as 3-D planes and evaluates the site specific probabilities of exceedance for given peak acceleration levels or pseudo-relative velocity levels. Based on a review of these data, and considering the relative seismic activity of the southern California region, a probabilistic peak horizontal ground acceleration of 0.70g was calculated. This value was chosen as it corresponds to a 10 percent probability of exceedance in 50 years (or a 475-year return period). Computer printouts of the FRISKSP program are included in Appendix C. Site specific response spectra can be provided under a separate cover upon request of the project structural engineer.

The following table lists the major faults and fault zones in southern California that could have a significant effect on the site should they experience activity.

FAULT ZONE	APPROXIMATE DISTANCE FROM SITE (Miles [Km])	FAULT ZONE	APPROXIMATE DISTANCE FROM SITE (Miles [Km])
Chino - Central Avenue	16.3 (26.2)	Puente Hills Blind Thrust	33.9 (54.5)
Clamshell - Sawpit	50.2 (80.8)	Raymond	50.0 (80.5)
Cleghorn	42.8 (68.9)	Rose Canyon	37.3 (60.0)
Coronado Bank	41.8 (67.2)	San Andreas - 1857 Rupture	44.1 (70.9)
Cucamonga	37.5 (60.3)	San Andreas - Mojave	44.1 (70.9)
Elsinore - Glen Ivy (Willard)	0.1 (0.1)	San Andreas - San Bernardino	34.5 (55.6)
Elsinore - Julian	28.6 (46.1)	San Jacinto - Anza	27.0 (43.5)
Elsinore - Temecula	2.7 (4.4)	San Jacinto - San Bernardino	26.0 (41.8)
Newport - Inglewood (LA Basin)	31.8 (51.2)	San Jacinto - San Jacinto Valley	23.0 (37.0)
Newport - Inglewood (offshore)	25.3 (40.7)	San Joaquin Hills	19.1 (30.7)
North Frontal Fault Zone (East)	51.2 (82.4)	San Jose	36.3 (58.4)
North Frontal Fault Zone (West)	41.4 (66.7)	Sierra Madre	39.5 (63.5)
Palos Verdes	41.4 (66.7)	Whittier	20.4 (32.8)
Pinto Mountain	46.5 (74.9)		

The possibility of ground shaking at the site may be considered similar to the southern California region as a whole. The relationship of the site location to these major mapped faults is indicated on the California Fault Map (see Figure 3). Our field observations and review of readily available geologic data indicate that known active faults cross the site.

It is important to keep in perspective that in the event of a "maximum probable" or upper bound ("maximum credible") 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. Life-safety is the intention of the code and recommendations herein. Given the relatively high anticipated ground acceleration, damage to planned facilities during significant earthquakes cannot be avoided or eliminated.



CALIFORNIA FAULT MAP
Jiron
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300
200
100
0
-100
-400
-300
-200
-100
0
100
200
300
400
500
600
SITE

Figure 3

Seismic Shaking Parameters

Based on the site conditions, Chapter 16 of the Uniform Building Code ([UBC], International Conference of Building Officials [ICBO], 1997) seismic parameters are provided in the following table:

Seismic Zone (per Figure 16-2*)	4
Seismic Zone factor Z (per Table 16-I*)	0.40
Soil Profile Types (per Table 16-J*)	S_D
Seismic Coefficient C_a (per Table 16-Q*)	0.44 N_a
Seismic Coefficient C_v (per Table 16-R*)	0.64 N_v
Near Source Factor N_a (per Table 16-S*)	1.3
Near Source Factor N_v (per Table 16-T*)	1.6
Distance to Seismic Source	0.1 mi. (0.1 km)
Seismic Source Type (per Table 16-U*)	B
Upper Bound Earthquake - Elsinore/Glen Ivy	M_W 6.8
Probabilistic Peak Horizontal Site Acceleration (10% probability of exceedance in 50 years)	0.7g
* Figure and table references from Chapter 16 of the UBC (ICBO,1997).	

SURFACE WATER AND GROUNDWATER

Surface

Surface water affecting the site is primarily limited to irrigation and precipitation. Runoff water from seasonal storms and/or from irrigation, which is not retained by either vegetation of soil, moves down-gradient via sheetflow toward Lake Elsinore. A Riverside County Flood Control District (RCFCD) drainage channel is located along the southeastern border of the property. Site development will require provisions for adequate control and disposal of surface water. The effects of surface flooding should be evaluated by the design engineer so that mitigative measures can be provided, if warranted. Due to the low plasticity and granular nature of the site soils, flooding and high velocity runoff across unpaved areas of the site may cause moderate to severe erosion.

Groundwater

Seeps, springs, or other indications of a high groundwater level were not noted on the subject property during the time of our field investigation. However, regional groundwater was encountered in all of the six (6) borings advanced onsite varying in depth from ±10 to ±15 feet in depth. In addition, pore pressure readings were taken in CPT-1, CPT-2, and CPT-3.

Due to the very gently sloping terrain (i.e., no canyons), and anticipated lack of flow line gradient at removal bottoms and a suitable outlet, the need for subdrains is generally not anticipated. However, should localized seepage be encountered during grading, recommendations for subdrainage would be made at that time. A leak detection, horizontal blanket drain is recommended in the pool area. The possibility of locally "perched" groundwater conditions exists due to the interbedded nature of the onsite alluvial materials. These observations reflect site conditions at the time of our investigation and do not preclude future changes in local groundwater conditions from excessive irrigation, precipitation, or that were not obvious, at the time of our investigation. The potential for perched water to occur during grading and after development exists, and should be anticipated. This potential should be disclosed to all interested parties, including homeowners and any homeowners association.

OTHER GEOLOGIC DEVELOPMENTAL CONSIDERATIONS

The effects of areal subsidence generally occur at the transition or boundaries between low-lying areas and adjacent hillside terrain, where materials of substantially different engineering properties (i.e., alluvium vs. bedrock) are present. Due to the gently sloping nature of the site, and the depth of the underlying bedrock, the potential for this condition is considered low. Based on the available data, the site is not situated directly on the boundary of an actively subsiding basin. Therefore, the potential for this phenomena to affect the site is also considered low, based on the available data.

In addition, our review of available data, as well as stereoscopic aerial photographs (USDA, 1980), showed no features generally associated with areal subsidence (i.e., radially-directed drainages flowing into a depression(s), linearity of depressions associated with mountain fronts, etc.) specifically on the site. Ground fissures are generally associated with excessive groundwater withdrawal and associated subsidence, or active faulting. Our review did not reveal any information that active faulting or excessive groundwater withdrawal, or ground fissures, or hydroconsolidation in the specific site vicinity, is occurring at this time. Therefore, the potential for areal subsidence or ground fissures is deemed low, based on the available data. However, groundwater levels may change in the future as a result of groundwater withdrawal/pumping, climatic change, or irrigation.

OTHER GEOLOGIC HAZARDS

Liquefaction, Lateral Spreading, and Dynamic Densification Potential

Seismically-induced liquefaction is a phenomenon in which cyclic stresses, produced by earthquake-induced ground motion, create excess pore pressures in soils. The soils may thereby acquire a high degree of mobility, and lead to lateral movement, sliding, sand boils, consolidation and settlement of loose sediments, 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. One key factor for initiation is that the site must experience a seismic event of a sufficient duration and magnitude, to induce straining of soil. Typically, liquefaction has a relatively low potential at depths greater than 50 feet.

Based on our review of the background data for the site, and our onsite field investigations, groundwater was evaluated to be ±10 to ±15 feet below existing ground surface (b.e.g.s.) For the purposes of this report, we have utilized a design regional water level of 11 feet b.e.g.s. in our liquefaction analyses, or an elevation of 1,253 to 1,264. Due to the remediated surface soils (±4 to ±7 feet below existing ground surface) and the lack of saturated soils in the upper ±7 feet at the time of our investigation, this elevation was considered reasonably conservative for the purpose of these analyses.

The condition of liquefaction has two principal effects. One is the consolidation of loose sediments with resultant settlement of the ground surface. The other effect is lateral sliding. Significant permanent lateral movement generally occurs only when there is significant differential loading, such as fill or natural ground slopes within susceptible materials. Differential loading conditions exist at the northeastern end the site, adjacent to Lake Elsinore. In the site area, we found there is a potential for seismic activity and a long-term groundwater table deeper than ±10 feet below the ground surface. Sediments underlying the site generally do not consist of medium- to fine-grained, cohesionless sands, but rather well-interbedded medium dense to dense, fine- to coarse-grained silty sand and clayey sand, with minor sand and clayey sand interbeds. Earth materials underlying the site are generally medium dense to dense where sandy, and stiff to hard where silty and clayey. Such materials are not generally prone to liquefaction. Due to their nature, the beds and lenses are interfingered and discontinuous, both horizontally and vertically, within the site area. Thus, GSI has concluded that liquefaction potential does not constitute a significant risk to site development beyond the established governing agency standards, provided our recommendations are implemented.

Seismic densification or volumetric strain of loose, relatively dry (much drier than optimum moisture), granular soils above the groundwater table may occur on this site when considering the seismic loading of the design basis earthquake. Potential densification was evaluated for soil profiles in our borings. For purposes of our review, the analyses (Appendix E) included the entire soil boring depth for the given earthquake and the assumption was that the regional groundwater was present at approximately 11 feet at the time of the design basis earthquake. These densification settlement magnitudes would be significantly reduced when considering the recommended remedial grading. If groundwater is present at the design elevation, the seismic densification analyses below that elevation would apply only to the earth materials between the fill soil and the groundwater depth (generally less than 5 feet), and the liquefaction analyses would be used to evaluate strains below the groundwater elevation (1,253 feet to 1,264 feet). Given the in place densities indicated in our borings and the results of our evaluations, the potential for seismic densification to impact the site following site development is considered low.

In accordance with the procedures in Special Publication 117 (CDMG, 1997), as well as the Recommended Procedures for Implementation of CDMG Special Publication 117 (SCEC, 1999), Guidelines for Analyzing and Mitigating Liquefaction Hazards in California, this report should be considered a "screening investigation," as provided for in Chapter 3 of Special Publication 117 and Section 4 of the Guidelines for Analyzing and Mitigating Liquefaction Hazards in California. Those guidelines state the following: *"The purpose of the screening investigations for sites within zones of required study is to filter out sites that have no potential or low potential for liquefaction."*

GSI has performed liquefaction analyses utilizing the "LiquefyPro" (Civil Tech, 2005) computer program to evaluate the liquefaction and densification potential of the site. The field and laboratory data obtained from our field exploration have been incorporated into the analyses. Results of the LiquefyPro program analyses are included in Appendix E of this report. The liquefaction analyses were performed on CPT-1 through CPT-11 using both the CPT data as well as the corresponding boring data from borings B-1 through B-6. The interbedded, relatively fine grained natural materials (alluvial fan deposits) were also evaluated using laboratory test data, including sieve analyses.

Our analysis used a design water depth of 11 feet b.e.g.s. Both "dry" sand densification and liquefaction were evaluated by GSI. The results were evaluated for non-mitigated ground conditions (i.e., current pre-graded conditions, and a water table that is present at a depth of 11 feet). Results of the seismic settlement above and below the design water level indicate up to approximately 1¼ inches of total settlement prior to grading. Following remedial grading of the surface soils, this magnitude is anticipated to be on the order of 1 inch of total settlement. Given the in-place densities of the fan deposits and the proposed grading, a correlation of 75 percent or less of the total settlement may be used on this site (SCEC, 1999). The resultant differential dynamic settlements would be within the County of Riverside guidelines of 2 inches in 40 lateral feet.

Based on the results of our site specific liquefaction analysis, GSI is of the opinion that the zones of sediments that may be susceptible to liquefaction beneath the site are within vertically and laterally discontinuous thin beds or lenses of sands. In addition, these beds and lenses are limited to localized areas. Therefore, it is our opinion that the potential for liquefaction at the study area is considered low. The potential for sand boils following earthwork and development is likewise considered low. The potential for seismic induced lateral spreading is considered low at this time due to the absence of existing fill or natural ground slopes with significant differential loading, as well as the planned shoreline mitigation. Our evaluation indicates that the potential for liquefaction and associated adverse effects within the site is generally low even considering a potential future rise in groundwater levels. The site conditions will also be improved by removal and recompaction of low density near-surface soils.

Given the depth of the liquefiable soils onsite, the proposed pool is considered to be the most susceptible improvement to seismic induced deformations. Mitigation of the liquefaction effects on the pool is presented in the following sections of this report. Lateral deformation of the fill and natural soils onsite, when subjected to strong seismic shaking, will be mitigated by the installation of a sheet pile walls along the lake side area of the project.

Accordingly, it is our opinion that the standards outlined in Special Publication 117 have been met and sufficient data exists to support, and is consistent with, the stated conclusions regarding the lack of potential hazard from liquefaction. Additionally review and/or analyses may be recommended, if or when site grading plans are finalized and/or revised.

Lateral Spreading

Lateral spread phenomenon is described as the lateral movement of stiff, surficial, mostly intact blocks of sediment displaced downslope towards a free face along a shear zone that has formed within the liquefied sediment. The resulting ground deformation typically has extensional fissures at the head of the failure, shear deformations along the side margins, and compression or buckling of the soil at the toe. The extent of lateral displacement typically ranges from half an inch to several feet. Two types of lateral spread can occur: 1) lateral spread towards a free face (e.g., drainage canal, lake, or shoreline embankment); and 2) lateral spread down a ground slope where a free face is absent. Factors such as earthquake magnitude, distance from the seismic energy source, thickness of the potentially liquefiable layers, and the fines content and particle size of those sediments also correlated with ground displacement.

In order for lateral spread to occur, a continuous liquefiable layer must exist parallel to the ground slope. As indicated previously, the isolated potentially liquefiable layers underlying portions of the site are not only vertically and laterally discontinuous. Due to the placement and subsequent confinement of onsite soils from the lakeshore by sheet piles, a free face condition and significant ground slopes do not exist, and the majority of the site is relatively flat lying. Therefore, it is our opinion that given the planned mitigation and development, seismically-induced lateral spread poses a low risk to the proposed site development.

Slope Stability

Based on our experience on similar projects, proposed fill slopes constructed using onsite materials should be grossly and surficially stable provided the recommendations contained herein are implemented during site development. Although unlikely, significant cut slopes, exposing poorly consolidated and/or sandy alluvial materials would likely require stabilization and/or erosion protection. All slopes should be designed and constructed in accordance with the minimum requirements of the County of Riverside and City of Lake Elsinore and the recommendations contained in Appendix F (General Earthwork and Grading Guidelines). Aside from the walls at the northern edge of the property and the pool wall, GSI does not anticipate cut or fill slopes in excess of ±5 feet in vertical height at this project.

Slope Considerations and Slope Design

All slopes should be designed and constructed in accordance with the minimum requirements of the County and City authorities, the recommendations in Appendix F, and the following:

1. Fill slopes should be designed and constructed at a 2:1 (horizontal to vertical [h:v]) gradient, or flatter, and not exceed about 15 feet in height. Fill slopes should be properly built in accordance with the design standards provided in the UBC/CBC (ICBO, 1997 and 2001) and compacted to a minimum relative compaction of 90 percent throughout, including the slope surfaces. Guidelines for slope construction are presented in Appendix F.

2. Cut slopes should be designed at gradients of 2:1 (h:v), or flatter, and not exceed about 15 feet in height. Significant cut slopes, exposing poorly consolidated and/or sandy alluvial materials would likely require stabilization and/or erosion protection.

3. Cut slopes should be mapped by the project geologist during grading to allow amendments to the recommendations, should exposed conditions warrant alternation of the design or stabilization.

LABORATORY AND FIELD TESTING

Classification

Soils were classified visually according to the Unified Soils Classification System and by supplemental index testing. The soil classifications are shown on the Boring Logs (see Appendix B).

Moisture-Density

The field moisture content and dry unit weight were determined for each undisturbed sample of the soils encountered in the borings. The dry unit weight was determined in pounds per cubic foot (pcf), and the field moisture content was determined as a percentage of the dry weight. The results of these tests are shown on the Boring Logs (Appendix B).

Laboratory Standard

The maximum dry density and optimum moisture content was determined for the major soil types encountered in the borings. The laboratory standard used was ASTM D-1557. The moisture-density relationship obtained for this soil is shown below:

SOIL TYPE	LOCATION AND DEPTH (FT.)	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE CONTENT (%)
Dark Brown, Silty SAND	B-2 @ 5 - 10'	134.5	8.5
Brown, Silty SAND	B-5 @ 5' - 8'	133.5	8.5

Expansion Testing

Expansion Index (E.I.) testing was performed on representative soil samples, in accordance with UBC Standard No. 18-2 of the 1997 UBC. Test results of E.I. = 7 indicate that onsite soils are anticipated to be generally very low expansive potential (E.I. from 0 to 20). Variations may occur, including soils exhibiting low and medium expansion potentials (E.I. from 21 to 90); additional E.I. testing should be performed during future development to further evaluate conditions encountered during our preliminary subsurface investigation.

Plasticity Index Testing/Atterberg Limits

For preliminary engineering design and foundation planning purposes, plasticity index testing/Atterberg Limits was performed on a selected sample from the onsite borings. Testing was performed to evaluate the liquid limit, plastic limit and plasticity index in general accordance with ASTM D-4318-64. The test results were also utilized to evaluate the soil classification in accordance with the Unified Soil Classification System. The laboratory test results are presented in Appendix D.

Sulfate/Corrosion Testing

Typical samples of the site materials were analyzed for soluble sulfates, pH, and resistivity. The soluble sulfate and corrosion potential results are shown as follows:

LOCATION AND DEPTH (FT.)	SOLUBLE SULFATES PERCENTAGE BY WEIGHT	pH	RESISTIVITY (OHMS-CM)
B-2 @ 5' - 10'	0.0066	7.2	8,000
B-5 @ 5' - 8'	0.0069	7.5	4,500

For preliminary planning purposes, based upon the soluble sulfate test results and the latest edition of the UBC/CBC, the soluble sulfate content is generally categorized as negligible (0.00-0.10 Water-Soluble Sulfate in Soil, percentage by weight) and sulfate-resistant concrete generally does not appear to be necessary, at this time. Additionally, based on the available data, a modified cement to water ratio and modified concrete compressive strength are not required for these conditions. Based on the results of the resistivity and pH testing, the onsite soils are judged to be generally neutral to mildly alkaline and are generally considered to be moderately corrosive toward ferrous metals in a saturated state. Chloride testing of the site soils indicates a low to non-detected level in the samples tested.

Although the site soils are categorized as being moderately corrosive to ferrous metals, no exposure conditions stated in Table 19-A-2 of the UBC/CBC (ICBO, 1997 and 2001) were encountered within the subject site on the soils tested. It is our understanding that ferrous metals embedded in properly poured and formed Type I, II, or V concrete should be adequately protected from these conditions. Based on the conditions encountered, a consulting corrosion engineer should be retained to provide specific recommendations for foundations, piping, etc.

Consolidation Testing

Consolidation testing was performed on relatively undisturbed soil samples in general accordance with ASTM Test Method D-2435-90. The consolidation test results are presented in Appendix D.

Sieve Analysis

Sieve analysis including passing a No. 200 (200 wash) tests were conducted on representative samples of the bedrock and alluvium encountered during our supplemental subsurface investigation. The laboratory test results are presented in Appendix D.

Shear Testing

Shear testing was performed on a representative sample in a direct shear machine of the strain-control type. The rate of deformation is approximately 0.05 inches per minute. The samples were sheared under varying confining loads in order to determine the Coulomb shear strength parameters, angle of internal friction and cohesion. In addition, an unconsolidated undrained triaxial shear test was performed on a sample from Boring B-1. The tests were performed on undisturbed samples of older alluvial deposits. The shear testing results are presented in Appendix D.

R-value

Representative samples of onsite materials were collected for Resistance Value (R-value) testing in general accordance with the California Materials Method No. 301. Preliminary test results indicate an R-value of 78. Preliminary pavement design is provided in the conclusions and recommendations section of this report. Additional R-value sampling should be performed during future development to verify conditions encountered during our preliminary subsurface investigation.

EARTHWORK BALANCE

Embankment factors (shrinkage) for the site have been estimated based upon our field and laboratory testing, visual site observations, and experience on nearby projects. 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, and discing. However, all these factors are difficult to define in a three-dimensional fashion.

Therefore, the information presented below represents average shrinkage/bulking values, using certain earthwork assumptions as follows:

Range of maximum density	=	133.5 to 134.5 pcf
Average Relative Compaction	=	92%

Undocumented Fill . . . . 7% - 10% shrinkage
Fan Deposits . . . . 5% to 13% shrinkage

An additional shrinkage factor item would include the removal of individual large plants or trees, or disturbed areas from demolition. 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 determine the net loss. Subsidence due to dynamic compaction from equipment routes, etc., is estimated to be 0.20 feet. Further, the degree of compaction attained by the grading contractor may also influence shrinkage by up to 5 percent. Field densities, moisture contents, and laboratory test data is included in this report in Appendices B and D.

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

Based on our field exploration, laboratory testing, and our engineering and geologic analyses, it is our opinion that the project site is suited for the proposed commercial use from a soils engineering and geologic viewpoint. Geotechnical developmental considerations include seismic shaking as a result of earthquakes on nearby and/or regional faults, thick alluvium and potential for associated differential settlements, potential for expansive soils, and corrosion potential. Regional seismic shaking, ranging from moderate to severe may occur, based on the proximity to active mapped earthquake faults. Accordingly, the proposed structures and foundations should be designed to resist seismic forces in accordance with the criteria contained in the UBC/CBC (ICBO, 1997 and 2001) for Seismic Zone 4. The recommendations presented above and below should be incorporated in the design, grading, and construction considerations.

General

1. Soils engineering and compaction testing services should be provided during future 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 verify and/or further evaluate geologic conditions. Although unlikely, if adverse geologic structures (i.e., faults, existing fill, ravelling sands exposed in slopes, etc.) are encountered, supplemental recommendations and earthwork may be warranted.

3. In general, groundwater is not expected to be a major factor in development of the site. However, dewatering may be necessary during remedial removals in the northeastern portion of the site at the location of the proposed pool. The dewatering will be dependant upon the final elevation of the pool, boat access ramp, and access road. Seeps and seasonal saturated soil zones not encountered at the time of our field work may be present and contingency for this condition should be considered, both during site work and after development.

4. It should be noted that if onsite soils are overly wet at the time of earthwork, drying of soils prior to compaction may be necessary.

5. Please note that although dewatering and recompaction efforts will mitigate the effects of settlement on the pool and appurtenances, the pool shell will be underlain by a liquefiable deposit 25 to 35 feet b.e.g.s. and therefore, will require pile supports to improve performance during seismic shaking, as well as reduce the potential for static vertical and horizontal movement.

6. Due to the nature of some of the onsite materials as well as long-term drying of cohesive materials, some caving and sloughing may be anticipated to be a factor in subsurface excavations and trenching. Therefore, current local and state/federal safety ordinances for subsurface trenching and other excavations should be implemented, and shoring may be necessary. All excavations should be performed in accordance with CAL-OSHA standards.

7. The materials excavated within the site are expected to be suitable for placement in compacted fills; however, they will generally need to be moisture conditioned near the surface in order to achieve at least optimum moisture content. Miscellaneous concrete or other debris remaining from demolition will require disposal offsite prior to grading. Any such debris remaining following these disposals would need to be isolated and removed from the fill prior to compaction efforts.

   The soil materials encountered throughout the site are expected to generally be very low in expansion potential; however, low to medium expansive soils can not be precluded. Based on preliminary test results, the soils generally have negligible sulfate contents per the UBC/CBC (ICBO, 1997 and 2001). Accordingly, the use of sulfate resistant concrete is not anticipated, based on the available data. Corrosion protection for buried metallic structures may be necessary. Accordingly, a consulting corrosion engineer should be retained to provide specific recommendations for foundations, piping, etc.

8. Cut slopes within fan deposits may require replacement and/or stabilization should loose and/or poorly consolidated materials be exposed. All proposed cut slopes should be evaluated for stabilization in the field, on a case by case basis.

9. General Earthwork and Grading Guidelines are provided at the end of this report as Appendix F. Specific recommendations are provided below.

Demolition/Grubbing

1. Existing shrubs, native grasses, and any miscellaneous trash and debris should be removed from the site, prior to grading.

2. Any previous foundations, or remnants of foundations, appurtenant structures, and improvements, cesspools, septic tanks, leach fields, or other subsurface structures

uncovered during the recommended removal and recompaction work should be observed by GSI so that appropriate remedial recommendations can be provided during grading.

3. Cavities, loose soils, soft soils, etc., remaining after demolition and site clearance should be cleaned out, observed by the geotechnical consultant's representative, processed, and replaced with fill which has been moisture conditioned to at least optimum moisture content and compacted to at least 90 percent of the laboratory standard (ASTM D-1557).

Treatment of Existing Ground

1. All existing undocumented fill on the site should be removed in areas planned for development. Undocumented fill was noted to a depth of at least ±9 feet in Boring B-1. Locally deeper areas may exist. These materials may be re-used as engineered fill. In the area near the proposed swimming pool, these removals may encounter groundwater associated with Lake Elsinore. As indicated above, this may necessitate local dewatering in order to achieve a suitable removal bottom. The depth of removals should be further evaluated during grading by the soils engineer.

2. Remedial removals in the older alluvial fan deposits across the site are generally anticipated to be on the order of about ±4 to ±7 feet with the potential for locally deeper removals (due to dry or porous materials), up to about ±15 feet. The field testing criteria for the base of remedial removals at the site is provided herein. The removal bottoms should be greater than or equal to about 85 percent saturation, and/or greater than or equal to about 105 pcf dry density for in-place native materials, which has been acceptable in various agency jurisdictions throughout various counties. The exposed removal surface should be scarified to a depth of 12 inches, moisture conditioned (if necessary), and then compacted prior to fill placement to finish pad grade. Again, localized deeper removals may be necessary due to buried drainage channel meanders, or dry, porous materials up to about ±15 feet deep. The project soils engineer/geologist should observe all removal areas during the grading.

3. On a preliminary basis, removals at the property boundaries will likely be restricted unless permission is granted by adjacent property owners to perform offsite remedial removals. Based on the anticipated removal depths, such offsite removals, for structural areas, would likely be approximately ±5 to ±7 feet of encroachment, for a 5-foot removal depth, into the adjacent properties. The scenarios below assume no offsite settlement sensitive structures exist within 5 feet of the property line. The offsite property to the north is occupied by a mobile home community. These properties will likely be subjected to equipment vibrations and noise regardless of what shoring or methods may be utilized.

Assuming offsite earthwork is not performed, onsite removals may consist of removal of ±5 feet in depth on a 1:1 projection from the property line into the project. On a preliminary basis, a structural setback of approximately ±10 feet, from the property lines would then be required. In lieu of setbacks, more conservative post-tension foundation or deep foundation (e.g., caissons) designs may be utilized for any site improvements. Design parameters for such foundation systems can be provided upon request. Depending upon actual removal depth and extent attained during grading, revised setbacks may be offered. Any settlement sensitive improvements constructed within the zone of limited or no removals may be subject to settlement and distress. This potential will need to be disclosed to all homeowners and any homeowners association.

GSI understands that the adjacent mobile home community will not allow grading to proceed onto their existing property. To accomplish the remedial grading, GSI recommends a temporary/permanent shoring wall be designed for this location. This will allow for remedial grading along the common property line to a lower (recommended) grade. The shoring should consist of H-piles and pre-fabricated concrete lagging. The H-piles should be embedded by placing them into drilled pier holes. Placement of H-piles by driving (hammering) is not recommended at this time. H-piles and drilled piers will be sized by the shoring/wall designer. However, the H-piles should be embedded into piers no smaller than 12 inches in diameter and should not be less than 2 feet of embedment for every foot of retained material if shoring is converted into a permanent wall.

4. Subsequent to the above removals, the exposed subsoils should be brought to <u>at least</u> optimum moisture content, then compacted to obtain a minimum relative compaction of 90 percent of the laboratory standard to a depth of 12 inches.

5. All earth materials may be reused as compacted fill provided that major concentrations of vegetation, oversized material (see Appendix F) and debris are removed prior to fill placement.

6. All grading should conform to the guidelines presented in Appendix F, the UBC/CBC (ICBO, 1997 and 2001), and the requirements of County and City authorities.

Fill Placement

1. Fill materials should be brought to <u>at least</u> optimum moisture, placed in thin, 6- to 8-inch lifts and mechanically compacted to obtain a minimum relative compaction of 90 percent of the laboratory standard.

2. Fill materials should be cleansed of major vegetation and debris prior to placement.

3. Although not anticipated, any oversized rock materials greater than 8 inches in diameter should be placed under the recommendations and supervision of the soils engineer. Utility trench standards dictate that smaller gravels (<2 inches) be used in backfill of trenches.

4. Any import materials should be inspected and determined suitable by the soils engineer prior to import or placement on the site. Foundation designs may be altered if the import materials have greater expansion/sulfate corrosion potentials than the materials encountered during our subsurface investigation.

Slope Stability

All slopes should be constructed in accordance with the minimum requirements of the County and City authorities, the UBC/CBC (ICBO, 1997 and 2001), and the Grading Guidelines presented in Appendix F. Based on our review, fill slopes are anticipated to perform adequately in the future with respect to gross and surficial stability, if the soil materials are maintained in a solid or semi-solid state and are limited to the heights prescribed herein. Cut slopes within alluvial deposits may require replacement and/or stabilization should loose and/or poorly consolidated materials be exposed. All slope faces should be considered erosive. All proposed cut slopes should be evaluated for stabilization in the field, on a case by case basis. Slopes should be planted/landscaped as soon as possible after construction.

Our opinion regarding the stability of the pool/sheet wall system will have to be reviewed once the final elevations of the pool are selected. The final configuration of the pool and adjacent sheet pile wall will have to be reviewed for local and global stability.

Fill Slopes

Any proposed fill slopes should be designed and constructed at gradients of 2:1 (h:v), or flatter, and not exceed about ±15 feet in height without additional analyses. Proposed fill slopes constructed to a maximum height of about ±15 would be considered satisfactory and should possess adequate factors of safety against instability, provided they are properly constructed and maintained.

Fill keys should be one-half the slope height in width or maintain a minimum width of 15 feet, whichever is greater. Fill keys should be excavated a minimum 2 feet in depth below proposed grades. Fill keys should also extend laterally, at a one to one (1:1) projection to the bottom of the removal surface, past the toe of the fill slope.

The importance of proper fill compaction to the face of slope cannot be overemphasized. In order to achieve proper compaction to the slope face, one or more of the four following methods should be employed by the contractor following implementation of typical slope construction guidelines: 1) track walk the slopes at grade; 2) grid roll the slopes; 3) use a combination of sheep foot roller and track walking; and/or, 4) overfill the slope 3 to 5 feet laterally and cut it back to grade. Random testing should be performed to verify compaction to the face of the slope. If the tests do not meet the minimum recommendation of 90 percent relative compaction, the contractor will be informed and additional compactive efforts recommended.

Cut Slopes

Any proposed cut slopes should be designed at gradients of 2:1 (h:v), or flatter, and should not exceed about ±15 feet in height. Based upon the materials types present and the relatively level to gently sloping topographic nature of the property as a whole, alluvial materials are anticipated to be exposed in cut slopes, if proposed; therefore, cut slopes within alluvial deposits will require replacement and/or stabilization should loose and/or poorly consolidated materials be exposed. For planning purposes keyways for any necessary stabilization fills should be a minimum of 15 feet in width and 2 feet in depth.

As with all grading projects, final evaluation of cut slopes during grading will be necessary in order to identify any areas of severely weathered, or otherwise adverse conditions. The need for remedial stabilization measures should be based on observations made during grading by a representative of this office. Based on our observations, specific remedial recommendations will be offered for stabilization, if warranted. Recommended designs for these systems are presented in Appendix F.

Temporary Construction Slopes

Due to the conditions expected to be encountered during rough grading, it is anticipated that temporary construction slopes for removals, backcuts, false slopes, haul roads, and other temporary conditions should be constructed at a maximum slope ratio of 1:1 (h:v), or flatter. Excavations for drainage devices, debris basins, and other localized conditions should be evaluated on an individual basis by the soils engineer and engineering geologist for variance from this recommendation. Due to the nature of the materials anticipated, the engineering geologist should observe all excavations and fill conditions. The geotechnical engineer should be notified of all proposed temporary construction cuts, and upon review, appropriate recommendations should be presented.

While the anticipated temporary slopes associated with removals along the property are anticipated to be generally stable, the possible instability of temporary cut slopes during removals or excavation cannot be precluded, and should be emphasized to the grading contractor. The temporary stability depends on many factors, including the slope angle, structural features in the earth materials, shearing strength along planes of weakness, height of the slope, groundwater conditions, and the length of time the cut remains unsupported and exposed to equipment vibrations and/or rainfall.

The possibility of temporary cut slopes failing during recommended removals, keyway excavations, etc., may be reduced by:

1. Minimizing the operations extent, in both duration and physical dimensions.
2. Limiting the length of a cut exposed to destabilizing forces at any one time.
3. Cutting no steeper than those backcut inclinations specified by the geotechnical consultant.
4. Avoiding operation of heavy equipment or stockpiling soil or building materials on or near the top of the excavation. All OSHA requirements with regard to excavation safety should be implemented by the grading contractor.
5. Provide temporary drainage and diversion barriers for the grading work to reduce the potential for ponding and erosion.

Erosion Control and Slope Construction

Fill and/or cut slopes will be subject to surficial erosion. Relatively loose, poorly cemented, low plasticity, sands and silty sands are most susceptible to erosion. Onsite earth materials have at least a moderate erosion potential. Slope designs should conform to the parameters established in the UBC/CBC (ICBO, 1997 and 2001), and/or County and City authorities.

PRELIMINARY RECOMMENDATIONS - FOUNDATIONS

General

The foundation design and construction recommendations are based on laboratory testing and engineering analysis of onsite earth materials by GSI. In the event that the information concerning the proposed development plan is not correct or any changes in the design, location, or loading conditions of the proposed structures are made, the conclusions and recommendations contained in this report 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 supersede design(s) by the project design engineer or civil engineer specializing in structural design. Upon request, GSI could provide additional consultation regarding soil parameters, as related to foundation design. Preliminary recommendations for foundation design and construction are presented below. Final foundation recommendations should be provided at the conclusion of grading based on laboratory testing of fill materials exposed at finish grade.

Foundation recommendations are distributed into three main categories: (1) Conventional slabs for very low expansive soils, without significant settlement, can be used on commercial or residential foundations; (2) Mat-type slabs that minimally comply with 1997 UBC Section 1815 can be used on low to high expansive soils with significant anticipated settlement can be used on residential foundations; (3) Post-tensioned foundations that minimally comply with 1997 UBC Section 1816 can be used in low to high expansive soils with significant anticipated settlement, can be used on residential and commercial buildings. In addition, pier supported foundations with structural floors typically used for heavily loaded structures with significant anticipated settlement, may also be utilized.

Conventional Foundation Design

1. Conventional spread and continuous footings may be used to support the proposed structures provided they are founded entirely in properly compacted fill and static and dynamic settlements are less than 1 inch and ½ inch in 40 lateral feet, respectively.

2. Analyses indicate that an allowable bearing value of 1,000 pounds per square foot (psf) for Buildings 5, 6, 7, and 8, and 1,500 psf for all other buildings may be used for design of footings which maintain a minimum width of 12 inches (continuous) and 24 inches square (isolated), and a minimum depth of at least 12 inches into the properly compacted fill. Isolated interior or exterior piers and columns should be founded at a minimum depth of 24 inches below the lowest adjacent ground surface, and tied to the main foundation in at least one direction. The bearing value may be increased by one-third for seismic or other temporary loads. This value may be increased by 20 percent for each additional 12 inches of embedment, to a maximum of 1,500 psf for Buildings 5, 6, 7, and 8, and 2,000 psf for all other buildings. No increase in bearing value for footing width is recommended. In addition, foundation depths and widths should be constructed per the UBC/CBC (ICBO, 1997 and 2001) guidelines.

3. All footings should be embedded a minimum 12 inches into properly compacted fill, as measured from the lowest adjacent grade.

4. All continuous footings should be minimally reinforced with two No. 4 rebars, one top and one bottom.

5. Interior columns may be supported on isolated footings tied together in one direction, provided the angular distortions and differential settlement criteria provided herein are incorporated into project planning and design by the project

recommendations provided herein. More onerous recommendations provided by the design engineers should take precedence over these minimum requirements.

When reviewing the interior building footings, the design engineer should utilize grade beams to control lateral drift of isolated column footings (interior or exterior), if the combination of friction and passive earth pressure will not be sufficient to resist lateral forces statically or seismically.

PRELIMINARY FLOOR SLAB DESIGN RECOMMENDATIONS COMMERCIAL BUILDINGS

Concrete slab-on-grade floor construction is anticipated. The following are presented as minimum design parameters for the slab, they are in no way intended to supersede design by a structural engineer. Design parameters for "Light Load Floor Slabs" do not account for concentrated loads (e.g., fork lifts, other machinery, etc.) and/or the use of freezers or heating boxes. Moisture migration through the slabs should be controlled, and the recommendations in the Moisture Transmission Mitigation section of this report should be followed.

Light Load Floor Slabs

The slabs in areas that will receive relatively light live loads (i.e., standard office loading) should be a minimum of 5 inches thick and be reinforced with No. 3 reinforcing bar on 18 inches 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 recommended minimum compressive strength of concrete is 2,500 pounds per square inch (psi).

The project design 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 design engineer should design the slabs in areas subject to high loads (machinery, forklifts, storage racks, etc., or above standard office loading). The Modulus of subgrade reaction (k-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-value (modulus of subgrade reaction) of 100 pounds per square inch per inch (psi/in) would be prudent to utilize for preliminary slab design. The preliminary R-value test completed (R-value = 78) and/or a plate load test may be used to calculate the preliminary k-value on near surface fill soils.

Concrete slabs should be at least 5½ inches thick and reinforced with at least No. 3 reinforcing bars placed 12 inches on center in two directions. Selection of slab thickness compatibility with anticipated loads should be provided by the structural design engineer. Heavily loaded concrete slabs should be underlain with a minimum of 4 inches of ¾ inch crushed rock (vibrated into place), or 4 inches of aggregate base materials (Class 2 aggregate base or equivalent) compacted to a minimum relative compaction of 95 percent. 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. The use of expansion joints in the slab should be considered. Concrete used in slab construction should be 560-C-3250. Spacing of expansion or crack control joints should be modified based on the footprint of the area to be heavily loaded. The use of Class A slab joints and/or steel dowels should be considered.

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

Subgrade Preparation

The subgrade material should be compacted to a minimum 90 percent of the maximum laboratory dry density. Prior to placement of concrete, the subgrade soils should be well moistened to at least optimum moisture content and verified by our field representative. Moisture migration through the slabs should be controlled, and the recommendations in the Moisture Transmission Mitigation section of this report should be followed.

PRELIMINARY POST-TENSIONED SLAB FOUNDATION SYSTEM RECOMMENDATIONS

Post-tensioned slab foundation systems should be used to support the proposed buildings where all existing fills have been reprocessed.

General

The information and recommendations presented in this section are not meant to supersede design by a registered structural engineer or civil engineer familiar with post-tensioned slab design or corrosion engineering consultant. Upon request, GSI could provide additional data/consultation regarding soil parameters as related to post-tensioned slab design during grading. The post-tensioned slabs should be designed in accordance with the Post-Tensioning Institute method. Alternatives to the Post-Tensioning Institute method may be used if equivalent systems can be proposed which accommodate the expansion potential and differential settlement noted for this site.

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 using the 1997 UBC Section 1816, based on design specifications of the Post-Tensioning Institute. The following table presents suggested minimum coefficients to be used in the Post-Tensioning Institute design method.

Thornthwaite Moisture Index	-20 inches/year
Correction Factor for Irrigation	20 inches/year
Depth to Constant Soil Suction	7 feet
Constant Soil Suction (pf)	3.6

The coefficients are considered minimums and may not be adequate to represent future worst case conditions, such as adverse drainage and/or improper landscaping and maintenance. The above parameters are applicable provided structures have gutters and downspouts and positive drainage 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, design values were obtained from figures or tables of the 1997 UBC Section 1816 and presented in the following table These values may not be appropriate to account for possible differential settlement of the slab due to other factors (i.e., fill settlement). If a stiffer slab is desired, higher values of y_m may be warranted. Although not anticipated, if at the conclusion of rough grading, expansion index testing indicates greater than 130 (very high) special design will be required.

EXPANSION POTENTIAL	BUILDING NOS. 1, 2, 3, 4, 9, AND COMMERCIAL	BUILDING NOS. 5, 6, 7, 8
e_m center lift	5.0 feet	5.5 feet
e_m edge lift	3.5 feet	4.5 feet
Y_m center lift	1.7 inches	3.8 inch
Y_m edge lift	0.75 inch	1.8 inch
Bearing Value [1]	1,500 psf	1,000 psf
Lateral Pressure	225 psf	225 psf
Subgrade Modulus (k)	125 pci/inch	75 pci/inch
Minimum Slab Thickness	5.0 inches	5.0 inches
Perimeter Footing Embedment [2]	18 inches	24 inches

[1] Internal bearing values within the perimeter of the post-tension slab may be taken as 1,500 psf for 1 foot wide by 1 foot deep footing. This may be increased by 300 psf for each foot of embedment below the bottom of the slab, to a maximum of 2,500 psf.
[2] As measured below the lowest adjacent compacted subgrade surface.

Subgrade Preparation

The subgrade material should be compacted to a minimum 90 percent of the maximum laboratory dry density. Prior to placement of concrete, the subgrade soils should be well moistened to at least optimum moisture content and verified by our field representative. Moisture migration through the slabs should be controlled, and the recommendations in the Moisture Transmission Mitigation section of this report should be followed.

Perimeter Footings and Pre-Wetting

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

Deepened footings/edges around the slab perimeter must be used to minimize non-uniform surface moisture migration (from an outside source) beneath the slab. The embedment depth is presented in Table 2. The bottom of the deepened footing/edge should be designed to resist tension, using cable or reinforcement per the structural engineer.

Floor slab subgrade should be at, or above the soils optimum moisture content to a depth of 12 inches for low (E.I. 0 to 50), 24 inches for medium (E.I. 51 to 90), and 36 inches for high to very high (E.I. 91 to 130), prior to pouring concrete, for existing soil conditions. Pre-wetting of the slab subgrade soil prior to placement of steel and concrete will likely be recommended and necessary, in order to achieve optimum or greater moisture conditions. This is discussed further herein. Soil moisture contents should be verified at least 72 hours prior to pouring concrete.

MOISTURE TRANSMISSION MITIGATION

General

Slab-on-grade floors for interior residential and commercial buildings should mitigate the transmission of water or water vapor by following the recommendations in the following sections. Given the location of the lake near the buildings, permanent groundwater level, potential for perched water onsite, GSI recommends that the following be used as a minimum. These recommendations are not meant to supercede those of a corrosion consultant, structural engineer, nor a moisture/waterproofing specialist.

1. Concrete utilized shall have a minimum compressive strength of 4,000 psi and a maximum water-cement ratio of 0.50, provided that the concrete mix is monitored and tested for minimum compliance during construction by special inspectors, and that proper finishing techniques are utilized.

2. Concrete slabs, including garages, shall be underlain by 2 inches of clean, washed sand (S.E. $\geq$30) above a 15 mil vapor retarder (ASTM E-1745 - Class A or Class B, per Engineering Bulletin 119, [Kanare, 2005]) installed per the recommendations of the manufacturer, including all penetrations (i.e., pipe, ducting, rebar, etc.). The manufacturer shall provide instructions for lab sealing, including minimum width of lap, method of sealing, and either supply or specify suitable products for lap sealing (ASTM E-1745), and per the UBC/CBC (ICBO, 1997 and 2001).

   ACI 302.1R-04 (2004) states: "If a cushion or sand layer is desired between the vapor retarder and the slab, care must be taken to protect the sand layer from taking on additional water from a source such as rain, curing, cutting, or cleaning. Wet cushion or sand layer has been directly linked in the past to significant lengthening of time required for a slab to reach an acceptable level of dryness for floor covering applications." Therefore, additional evaluations may be necessary for the cushion or sand layer for uniformity (i.e., moisture content, sand thickness, etc.), prior to the placement of concrete.

3. The vapor retarder shall be underlain by 4 inches of gravel placed directly on the prepared, moisture conditioned, subgrade and should be sealed to provided a continuous retarder under the entire slab, as discussed above. The gravel shall meet the following minimum gradation requirements:

SIEVE SIZE[1] STANDARD	METRIC	½" AGGREGATE % PASSING
¾"	19mm	100
½"	12.5mm	71
⅜"	9.5mm	12
#4	4.75mm	1
#200	0.075mm	0 - 2
OTHER SPECIFICATIONS		
Sand Equivalent (ASTM E2419)		$\geq$35
Cleaness Value (CAL TEST 227)		75
Apparent Specific Gravity (ASTM C127)		2.65
Percent Absorption (ASTM C127)		1.2
Resistance to Abrasion (ASTM C131)		
% Loss After 100 Revolutions		4.7
% Loss After 500 Revolutions		20.4
Soundness (ASTM C88)		
Total % Loss By Weight		3.6
Test Method for Unit Weight (ASTM C29)[2]		86 lbs/ft^3
Rodding Procedure		2,322 lbs/yd^3
		1,16 tons/yd^3

1. Particle size (sieve) analysis (ASTM C136 & C117), percent passing
2. Test Method ASTM C29 should be verified on the date of actual production or prior to shipment to project site.

4. Additional recommendations regarding moisture protection from the project structural engineer and project architect should be provided.

5. For soils with a very low to medium E.I. within the top 7 feet, the finish grade concrete slabs, including garage areas, should be reinforced with No. 3 rebars at 18 inches on center, each way or equivalent steel, welded-wire mesh, provided the reinforcement is placed on chairs, at slab mid-height. The structural engineer for the building foundations and slab system should specify the equivalent steel mesh reinforcement. All slab reinforcement should be supported on chairs to ensure proper positioning at mid-height in the slab during placement of concrete. Hooking

of reinforcement is not an acceptable method of reinforcement placement. Garage slabs should be poured separately from living area footings. A positive separation should be maintained with expansion joint material to permit relative movement.

6. Pre-moistening and/or presaturation of the slab subgrade soil is recommended for these soil conditions on a preliminary basis. The moisture content of the subgrade soils should be equal to or greater than optimum moisture to a depth equivalent to the exterior footing depth in the slab areas (12 to 18 inches). Pre-moistening and/or presaturation should be evaluated by the soils engineer 72 hours prior to visqueen placement.

PRELIMINARY SETTLEMENT EVALUATION

The site is underlain by alluvial fan deposits that locally (at depth) display minor collapse potential (and thus, settlement) upon saturation. The calculated estimate for the total settlement due to consolidation and/or response to wetting (hydrocollapse) of the underlying soil as well as soil consolidation has been evaluated. The actual timing of this settlement is a function of when, and if, these materials actually become saturated. In addition, localized post-construction settlement due to the load of proposed structures will be low magnitude between similar foundations elements and will primarily be completed as the buildings are finished.

Therefore, the structures should be evaluated and designed for the combination of the soil parameters presented above and the estimated static and dynamic settlements and angular distortion values over 40 feet, provided below:

BUILDING NUMBER	*ESTIMATED TOTAL SETTLEMENT (INCHES)	*ESTIMATED DIFFERENTIAL SETTLEMENT (INCHES)	*ESTIMATED ANGULAR DISTORTION
1 through 9	2¼	1½	1/320
Commercial	1½	1	1/480

* Estimated static and dynamic settlements

In addition to the above, the structural engineer should also consider estimated settlements due to short duration seismic loading and applicable load combinations, as required by County and City authorities, and/or the UBC/CBC (ICBO, 1997 and 2001). The structural engineer should determine whichever condition controls the design. These estimates are based on fill materials that have generally very low to medium expansion potential, and therefore should be considered as preliminary estimates. Final settlement estimates should be provided at the completion of rough grading, based on the actual planned fill thickness and the nature of underlying materials encountered.

If determined by the project design engineer that the estimated settlements could compromise the structural integrity of the foundation system or superimposed structure, alternative remedial recommendations should be sought from this office. If the project design engineer determines that a stiffened conventional foundation system can tolerate the parameters outlined above, this office should review the foundation plans.

Slope Setback Considerations for Footings

Footings should maintain a horizontal distance, X, between any adjacent descending slope face and the bottom outer edge of the footing. The horizontal distance, X, may be calculated by using $X = h/3$, where h is the height of the slope. X should not be less than 7 feet, nor need not be greater than 40 feet. X may be maintained by deepening the footings.

PRELIMINARY PAVEMENT DESIGN RECOMMENDATIONS

General

The governing agency may retain the authority to approve the final structural design sections after subgrade elevations and actual R-values have been obtained at the conclusion of earthwork.

The preliminary pavement sections presented herein are based on a preliminary R-value obtained for typical site materials (R-value = 78), grading observations completed by GSI on other nearby properties, an assumed traffic index (TI) of 5.0, and the guidelines presented in the latest revision to the California Department of Transportation "Highway Design Manual" fifth edition. Based on this information, it is likely that pavement sections will likely be 3 inches of Asphaltic Concrete (A.C.) on 6 inches of Class 2 aggregate base rock (or equivalent). Final pavement designs should be based upon the TI provided by the design engineer and actual R-value testing of materials exposed at subgrade elevations at the conclusion of earthwork. The upper 12 inches of pavement section including supporting subgrade soils should be compacted to 95 percent relative compaction (per ASTM D-1557).

WALL DESIGN PARAMETERS

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. Building walls, below grade, should be water-proofed. 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. Recommendations for specialty walls (i.e., crib, earthstone, geogrid, etc.) can be provided upon request, and would be based on site specific conditions.

Restrained 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 pounds per cubic foot (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.

Cantilevered Walls

The recommendations presented below are for cantilevered retaining walls up to 10 feet high. Design parameters for walls less than 3 feet in height may be superceded by City and/or 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 <u>do not</u> 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.

Retaining Wall Seismic Surcharge

If retaining walls for the planned facility are over 5 feet or have the potential (during a seismic event) to fail and reduce/eliminate entry or exit to the facility, a seismic surcharge should be added to the wall pressures. This seismic surcharge (seismic increment) should be added as 15H, where "H" is the height of the wall (excluding the shear key). This is uniform pressure that surcharges the wall similar to a traffic load.

SURFACE SLOPE OF RETAINED MATERIAL (HORIZONTAL:VERTICAL)	EQUIVALENT FLUID WEIGHT P.C.F. (SELECT BACKFILL)	EQUIVALENT FLUID WEIGHT P.C.F. (NATIVE BACKFILL)
Level*	35	45
2 to 1	50	60
* 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.		

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). 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). Materials with an E.I. potential of greater than 65 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).

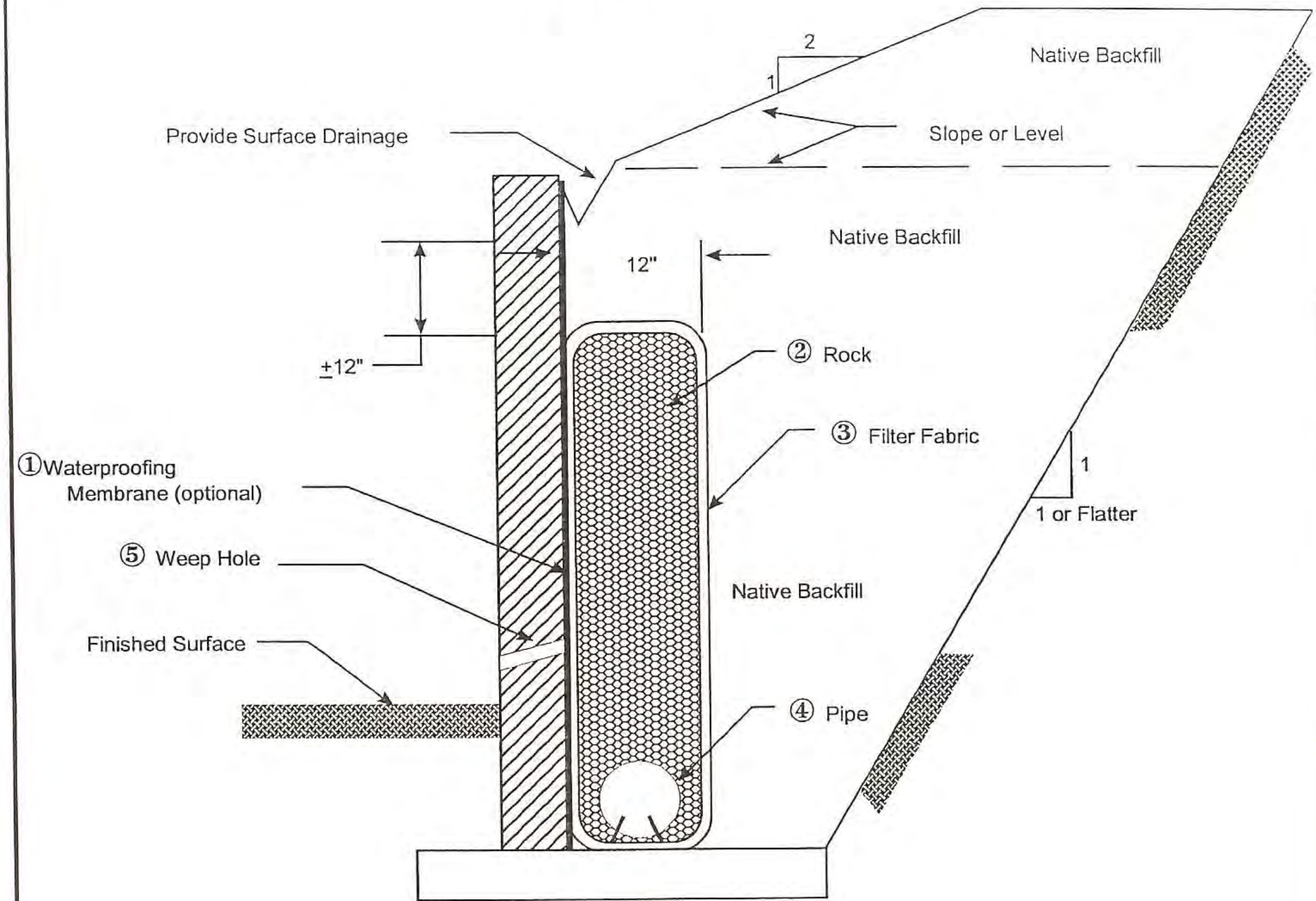
Outlets should consist of a 4-inch diameter solid PVC or ABS pipe spaced no greater than ±100 feet apart, with a minimum of two outlets, one on each end. The use of weep holes, only, in walls higher than 2 feet, is not recommended. The surface of the backfill should be sealed by pavement or the top 18 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:

DETAILS

N.T.S.



① **WATERPROOFING MEMBRANE (optional):**
Liquid boot or approved equivalent.

② **ROCK:**
3/4 to 1-1/2" (inches) rock.

③ **FILTER FABRIC:**
Mirafi 140N or approved equivalent; place fabric flap behind core.

④ **PIPE:**
4" (inches) diameter perforated PVC. schedule 40 or approved alternative with minimum of 1% gradient to proper outlet point (Perforations down).

⑤ **WEEP HOLE:**
Minimum 2" (inches) diameter placed at 20' (feet) on centers along the wall, and 3" (inches) above finished surface (No weep holes for basement walls.).

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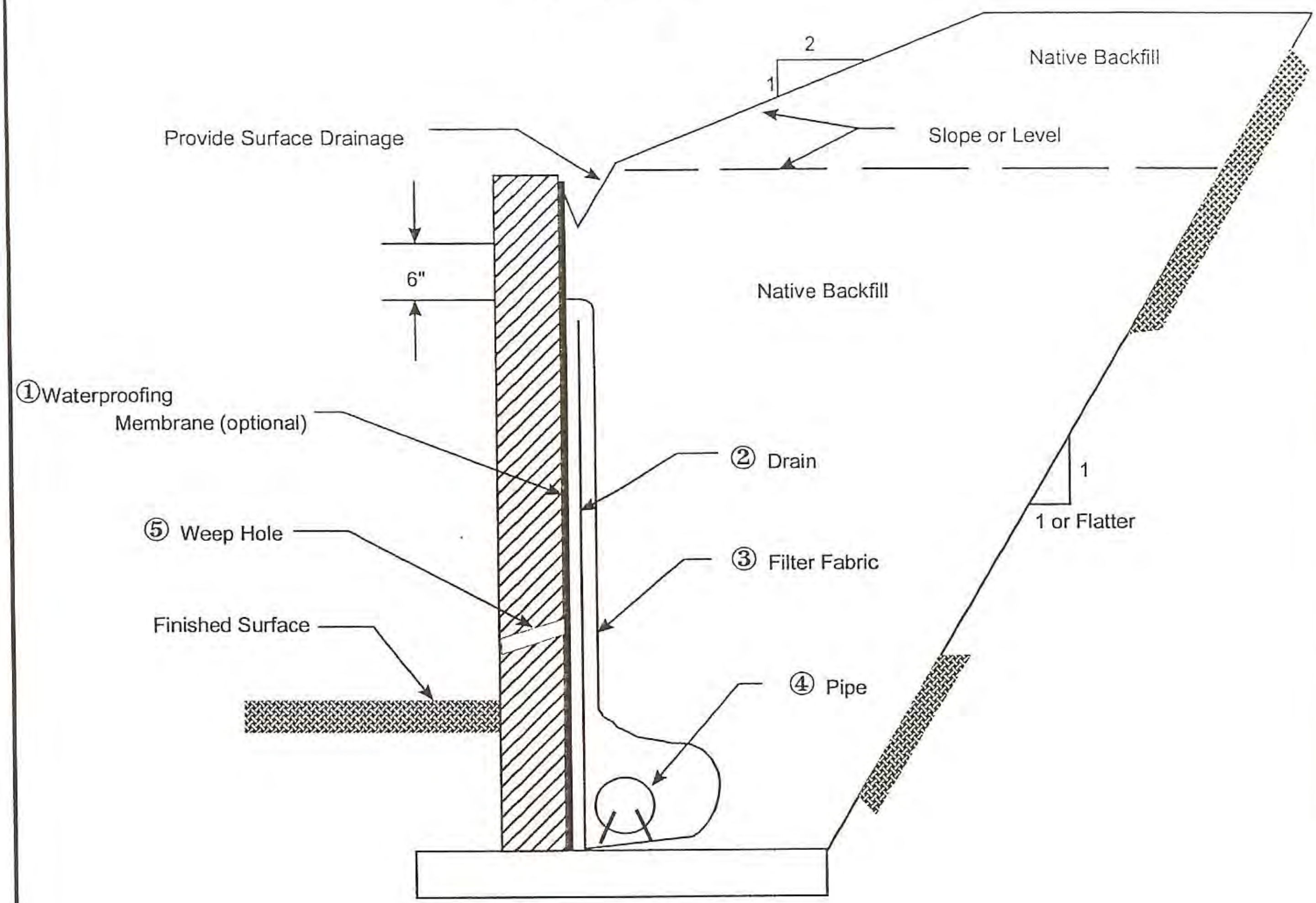
TYPICAL RETAINING WALL BACKFILL AND DRAINAGE DETAIL

DETAIL 1

Geotechnical • Coastal • Geologic • Environmental

DETAILS

N.T.S.



① **WATERPROOFING MEMBRANE (optional):**
Liquid boot or approved 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" (inches) diameter perforated PVC. schedule 40 or approved alternative with minimum of 1% gradient to proper outlet point.

⑤ **WEEP HOLE:**
Minimum 2" (inches) diameter placed at 20' (feet) on centers along the wall, and 3" (inches) above finished surface. (No weep holes for basement walls.)

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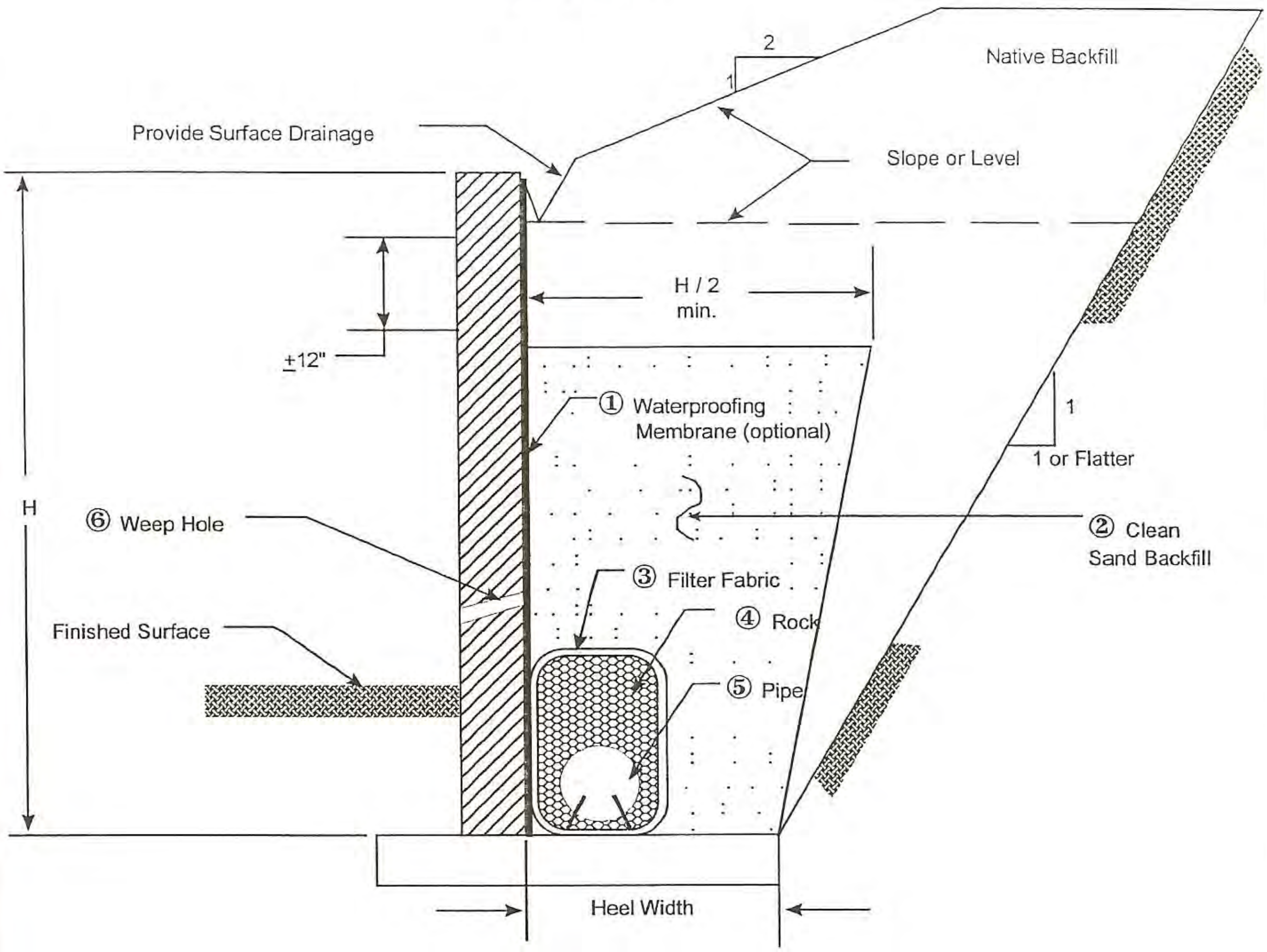
RETAINING WALL BACKFILL AND SUBDRAIN DETAIL GEOTEXTILE DRAIN

DETAIL 2

Geotechnical • Coastal • Geologic • Environmental

DETAILS
N . T . S .



① **WATERPROOFING MEMBRANE (optional):**
Liquid boot or approved equivalent.

② **CLEAN SAND BACKFILL:**
Must have sand equivalent value of 30 or greater; can be densified by water jetting.

③ **FILTER FABRIC:**
Mirafi 140N or approved equivalent.

④ **ROCK:**
1 cubic foot per linear feet of pipe or 3/4 to 1-1/2" (inches) rock.

⑤ **PIPE:**
4" (inches) diameter perforated PVC. schedule 40 or approved alternative with minimum of 1% gradient to proper outlet point (Perforations down).

⑥ **WEEP HOLE:**
Minimum 2" (inches) diameter placed at 20' (feet) on centers along the wall, and 3" (inches) above finished surface. (No weep holes for basement walls.)

GeoSoils, Inc.	RETAINING WALL AND SUBDRAIN DETAIL CLEAN SAND BACKFILL
	DETAIL 3
	Geotechnical • Coastal • Geologic • Environmental

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

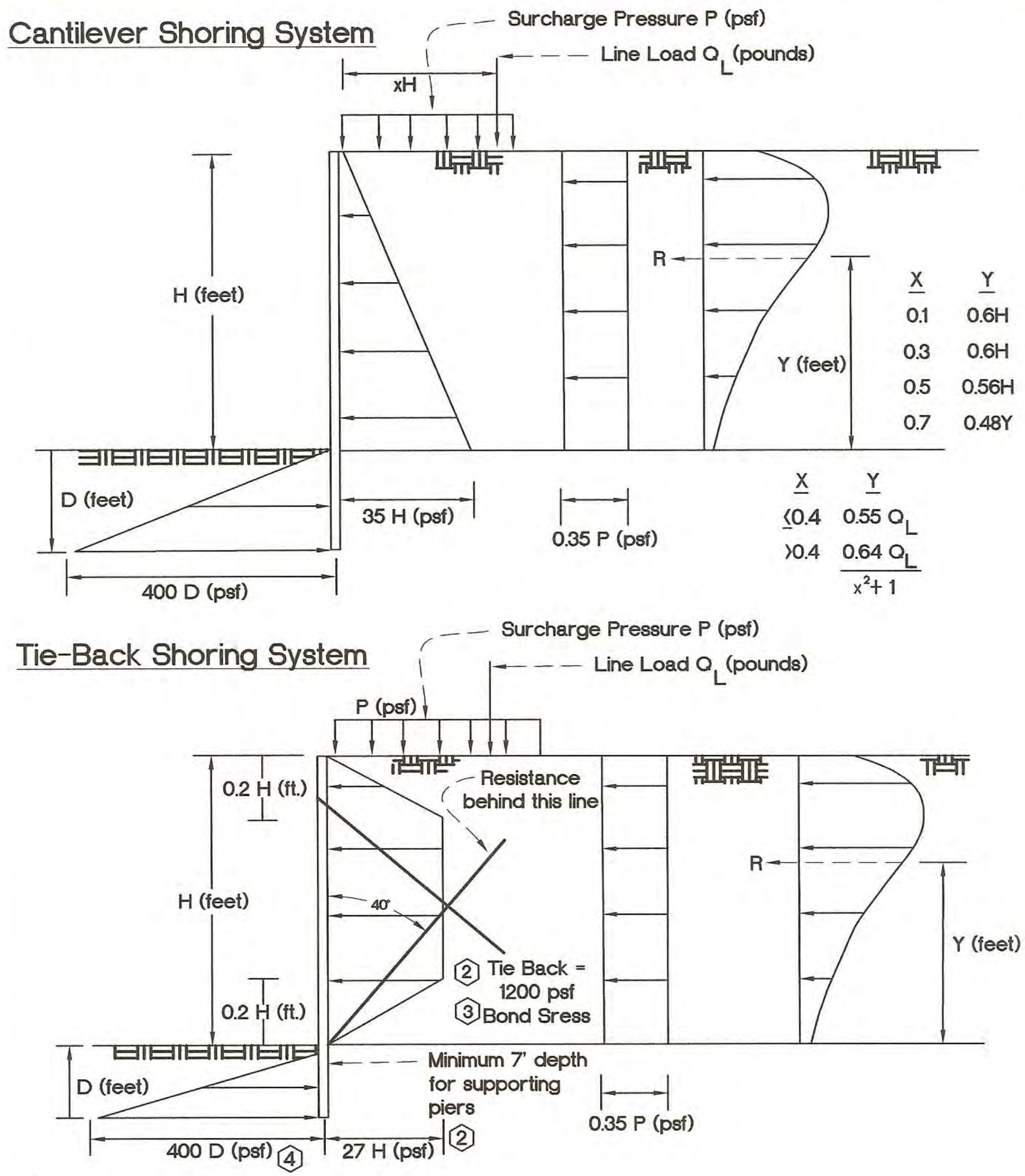
b) Increase of the amount of reinforcing steel and wall detailing (i.e., expansion joints or crack control joints) such that a angular distortion of 1/360 for a distance of 2H on either side of the transition may be accommodated. Expansion joints should be placed no greater than 20 feet on-center, in 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 formational 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.

Shoring of Excavations

Due to the relatively shallow ground water condition, and because of limited space and adjacent existing improvements (i.e., existing mobile home community), temporary shoring of vertical excavations along the northern site boundary of 4 to 7 feet will likely be required. We recommended that temporary slopes be retained either by a cantilever shoring system (limited to about 15 feet) deriving passive support from cast-in-place soldier piles (H-pile and lagging-shoring system) or a restrained tie-back and pile system. Based on our experience with similar projects, if lateral movement on the order of ½ to 1 inch of the shoring system, or height exceeding 15 vertical feet of retained material cannot be designed for or tolerated, we recommend the utilization of an internal bracing/raked shoring system. Shoring of excavations of this size is typically performed by specialty contractors with knowledge of local area soil conditions. We recommend that shoring contractors provide the excavation shoring design. However, for the shoring design parameters, we have provided Figure 4, Lateral Earth Pressures for Shoring Systems. Please note, the use of anchors may not be feasible on this site due to the location of adjacent existing improvements. If desired, additional anchor recommendations will be provided. Since design of retaining is sensitive to surcharge pressures behind the excavation, we recommend that this office be consulted if unusual load conditions are anticipated. Care should be exercised when excavating into the on-site soils since caving or sloughing of these materials is possible. Field testing of tie-backs (if used) and observation of soldier pile excavations should be performed during construction. Shoring of the excavation is the responsibility of the contractor. Extreme caution should be used to minimize damage to existing pavement and utilities caused by settlement or reduction of lateral support. Accordingly, we recommend that the foundations of adjacent structures be surveyed prior to and during construction to evaluate the effects of shoring on these structures. Photo-documentation of pre-construction conditions is also advised.



NOTES

1. Include groundwater effects below groundwater level.
2. Include water effects below groundwater level.
3. Grouted length greater than 7 feet; field test anchor strength.
4. Neglect passive pressure below base of excavation to a depth of one pier diameter.

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LATERAL EARTH PRESSURES
FOR SHORING SYSTEMS

Figure 4

W.O. 5043-A-SC DATE 05/06 SCALE None

Open Excavations

Construction materials and/or stockpiled soil should not be stored within 5 feet of the top of any temporary slope or trench wall. Temporary/permanent provisions should be made to direct any potential runoff away from the top of temporary excavations. It is the responsibility of the general contractor and his subcontractor to provide a safe working environment and to protect site improvements as well as adjacent existing improvements during construction.

Lateral Pressure

The active pressure to be utilized for trench wall shoring design may be computed by the rectangular active pressure psf as shown in Table 1. Passive pressure may be computed as an equivalent fluid having a given density shown in the following table:

Earth Pressure for Shoring (Level Ground Surface)

SOIL TYPE	RECTANGULAR ACTIVE PRESSURE (PSF)	EQUIVALENT FLUID WEIGHT FOR PASSIVE PRESSURE (PCF)
Alluvial Fan Deposits	35	200

The above criteria assumes that hydrostatic pressure is not allowed to build up behind excavation walls. If water is allowed to accumulate behind walls, an additional hydrostatic pressure surcharge should be added.

These recommendations are for excavation walls up to 20 feet high. Active earth pressure may be used for trench wall design, provided the wall is not restrained from minor deflections. An empirical equivalent fluid pressure approach may be used to compute the horizontal pressure against the wall. Appropriate fluid unit weights are provided for specific slope gradients of the retained material: these do not include other superimposed loading conditions such as traffic, structures, seismic events, expansive soils or adverse geologic conditions.

For excavation walls greater than 6 feet in height, a seismic increment of 10H (uniform pressure) may be considered for level excavation. For walls, these seismic loads should be applied at 0.6H up from the bottom of the wall to the height of retained earth materials.

Excavation Observation and Monitoring (All Excavations)

When excavations are made adjacent to an existing structure (i.e., utility, road or building) there is a risk of some damage to that structure even if a well designed system of excavation and/or shoring, is planned and installed. We recommend, therefore, that a systematic program of observations be made before, during, and after construction to determine the effects (if any) of construction on the existing structures.

We believe that this is necessary for two reasons: First, if excessive movements (i.e., more than one-half inch) are detected early enough, remedial measures can be taken which could possibly prevent serious damage to the existing structure. Second, the responsibility for damage to the existing structure can be determined more equitably if the cause and extent of the damage can be determined more precisely.

Monitoring should include the measurement of any horizontal and vertical movements of both the existing structures and the shoring and/or bracing. Locations and type of the monitoring devices should be selected as soon as the total shoring system is designed and approved. The program of monitoring should be agreed upon between the project team, the site surveyor and the Geotechnical Engineer of Record, prior to excavation.

Reference points on the existing structures should be placed as low as possible on the exterior walls of buildings adjacent to the excavation. Exact locations may be dictated by critical points within the structure, such as bearing walls or columns for buildings; and surface points on roadways and sidewalks near the top of the excavation. The points on the shoring should be placed under or very near the points on the structures.

For a survey monitoring system, an accuracy of a least 0.01 foot should be required. Reference points should be installed and read initially prior to excavation. The readings should continue until all construction below ground has been completed and the backfill has been brought up to final grade.

The frequency of readings will depend upon the results of previous readings and the rate of construction. Weekly readings could be assumed throughout the duration of construction with daily readings during rapid excavation near the bottom and at critical times during the installation of shoring or support. The reading should be plotted by the Surveyor and then reviewed by the Geotechnical Engineer.

In addition to the monitoring system, it would be prudent for the Geotechnical Engineer and the Contractor to make a complete inspection of the existing structures both before and after construction. The inspection should be directed toward detecting any signs of damage, particularly those caused by settlement. Notes should be made and pictures should be taken where necessary.

Observation

It is recommended that all excavations be observed by the geologist and/or geotechnical engineer. Any fill which is placed should be approved, tested, and verified if used for engineered purposes. Temporary trench excavations should be observed by the geologist and/or geotechnical engineer. Should the observation reveal any unforseen hazard, the geologist or geotechnical engineer will recommend treatment. Please inform us at least 24 hours prior to any required site observation.

Monitoring Existing, Offsite Improvements

It is recommended that existing, offsite improvement be inspected prior to the start of earthwork and be monitored during and at the conclusion of grading to determine if earthwork at the site has influenced the stability of said improvements. Pressure testing of nearby gravity, as well as pressured, utilities may be warranted following development to evaluate if site construction has interrupted these buried improvements.

Dewatering

As indicated above, removal depths will be on the order of ±4 to ±7 feet below the existing grade. During our field investigation groundwater was encountered at depths on the order ±10 to ±15 feet below the existing grade. Proposed removal excavations over most of the site should not encounter groundwater. However, removals in the northeast portion of the site (i.e., near the lake) are anticipated to encounter groundwater at about lake level. Therefore, dewatering efforts in this area appear to be necessary at this time. It is recommended that the Client and/or his representative(s) consult with a licensed dewatering contractor for the most efficient and cost effective design.

Sheet Piling

To reduce the effects of waves, lake level variations, seismic loading on the improvements near the edge of the lake, a sheet pile wall system is recommended. The tip elevation of the sheets should extend, if possible, to below elevation 1,227 feet MSL. This elevation was selected due to the potentially liquefiable sand layers evaluated in the alluvial fan deposits. Due to the location of the pool behind the sheet pile wall, sheets should be of the interlocking Z-type (cross section) and cantilever design. A secondary set of sheets is recommended for edge of property support about 15 feet lower in elevation that the first, also with a tip elevation of about 1,227 feet MSL.

Due to the location of existing improvements, these sheet piles should be placed by a vibratory pile hammer. Pre-drilling should not extend below 15 feet from the planned finished grade. Sheet pile design should proceed with trial sections of PDA 27 or PZ 32 (Grade 60 steel). Alternative sections using plastic materials in lieu of steel sheets should be evaluated by the sheet pile designer.

To assist the designer, sheet pile design data is included herein (US Steel, 1975). The conditions for tip embedment into purely granular soil and cohesive soils are given in Figures 5 and 6. A schematic of the sheet pile wall system at the pool/boat ramp area is provided in Figure 7. Final design for the pool/boat ramp area should be provided by the project design civil engineer and reviewed by GSI. Tie-backs may be needed (as depicted in Figure 7) in the design of the sheet piles if the sheet piles cannot be purely cantilevered or if this lateral support is needed in the pool design. Additional recommendations for tie-backs will be provided , if needed, in the final design(s).

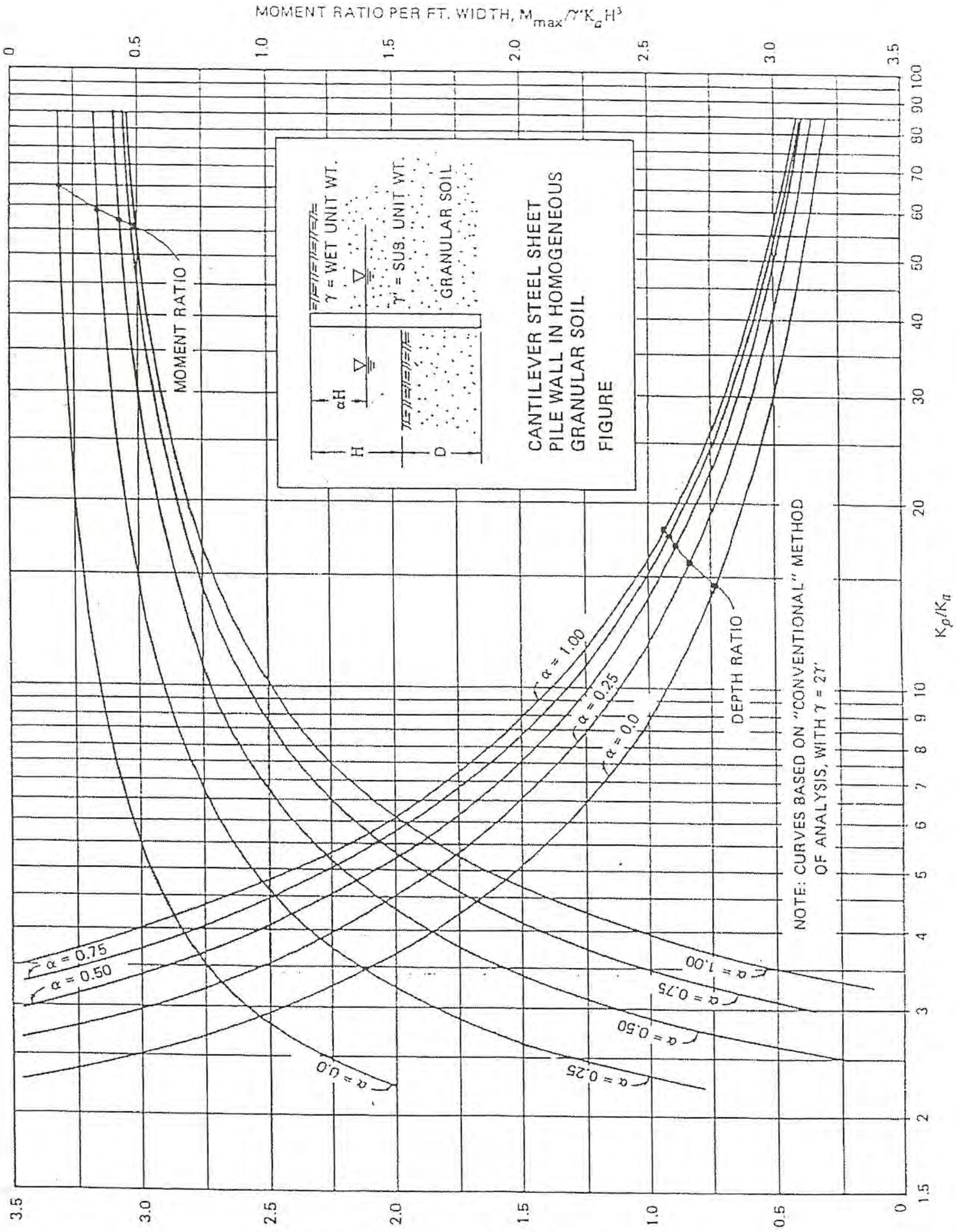
TOP-OF-SLOPE WALLS/FENCES/IMPROVEMENTS

Slope Creep

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 swimming pools, 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 homeowners and homeowners association.

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, or a combination of grade beam and caisson foundations, for expansion indices greater than 50 comprising the slope, with creep forces taken into account. The grade beam should be at a minimum of 12 inches by 12 inches in cross section, supported by



MOMENT RATIO PER FT. WIDTH, $M_{max}/\gamma' K_a H^3$
DEPTH RATIO, D/H (INCREASE BY 20% TO 40% FOR S.F.)
K_p/K_a
CANTILEVER STEEL SHEET PILE WALL IN HOMOGENEOUS GRANULAR SOIL
FIGURE
γ = WET UNIT WT.
γ' = SUB. UNIT WT.
GRANULAR SOIL
αH
H
D
MOMENT RATIO
DEPTH RATIO
NOTE: CURVES BASED ON "CONVENTIONAL" METHOD OF ANALYSIS, WITH $\gamma = 2\gamma'$
α = 1.00
α = 0.75
α = 0.50
α = 0.25
α = 0.0

Figure 5

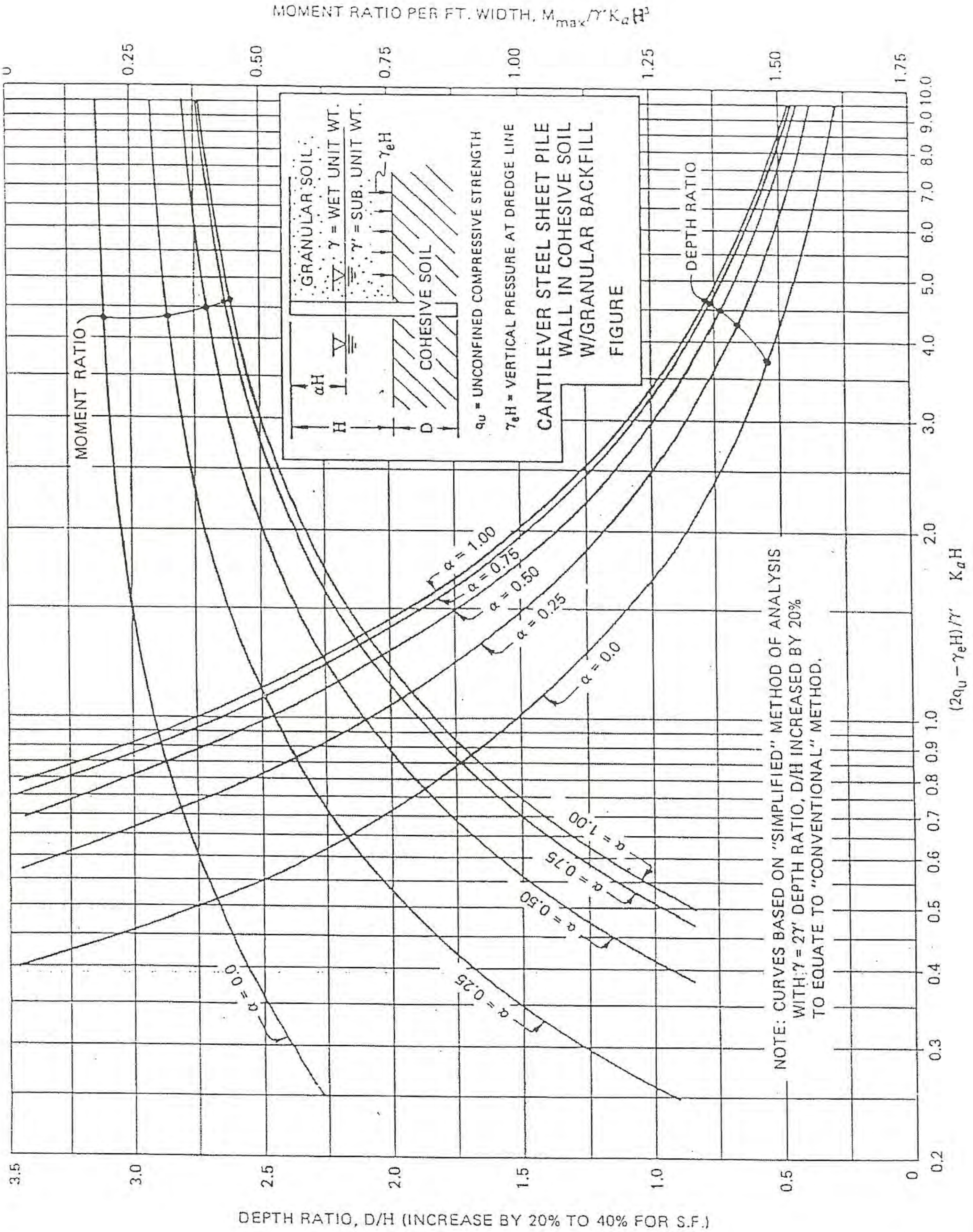
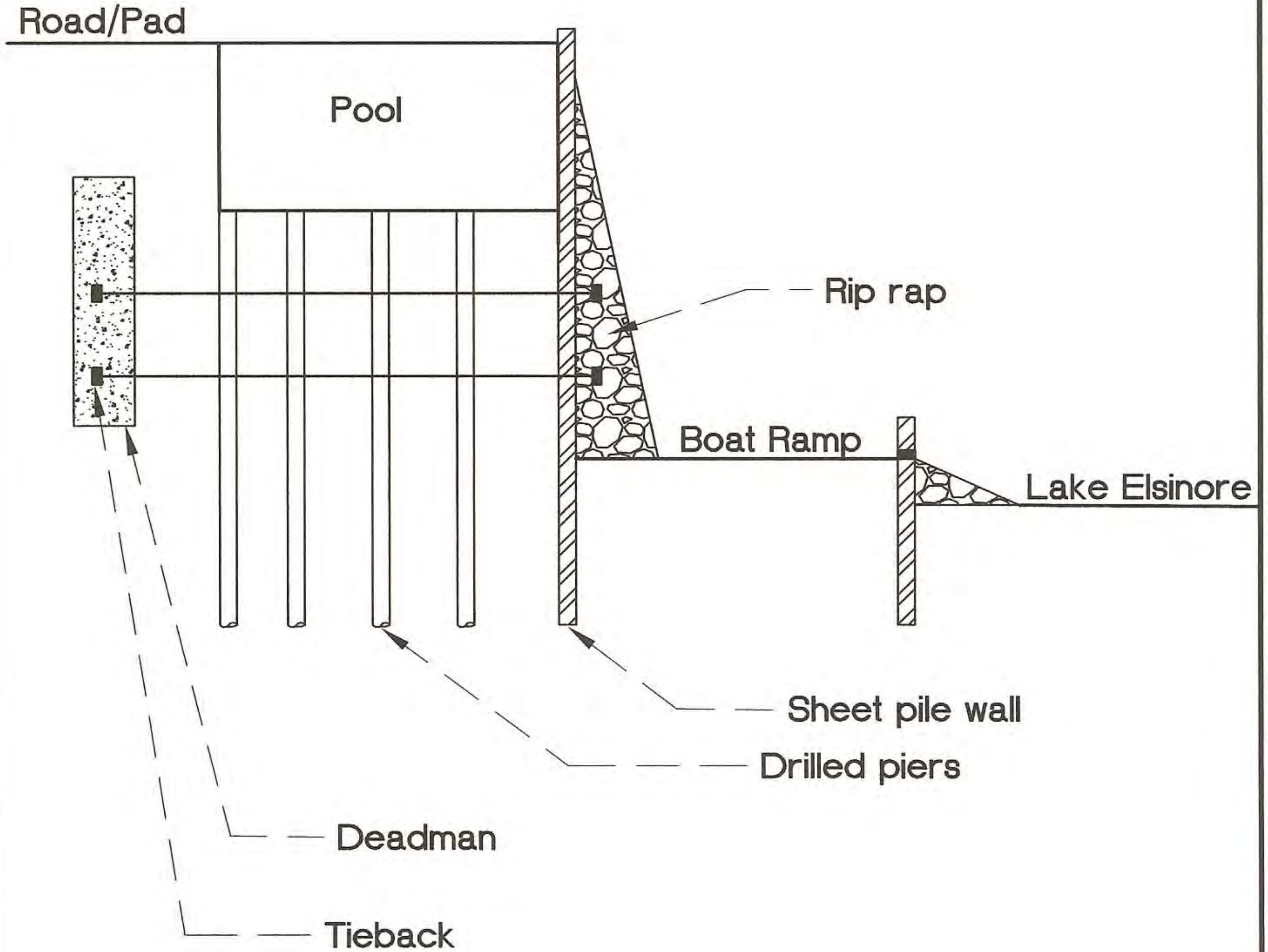

MOMENT RATIO PER FT. WIDTH, $M_{max}/\gamma' K_a H^3$
0
0.25
0.50
0.75
1.00
1.25
1.50
1.75
GRANULAR SOIL
γ = WET UNIT WT.
γ' = SUB. UNIT WT.
$\gamma_e H$
COHESIVE SOIL
αH
H
D
q_u = UNCONFINED COMPRESSIVE STRENGTH
$\gamma_e H$ = VERTICAL PRESSURE AT DREDGE LINE
CANTILEVER STEEL SHEET PILE WALL IN COHESIVE SOIL W/GRANULAR BACKFILL
FIGURE
MOMENT RATIO
DEPTH RATIO
α = 1.00
α = 0.75
α = 0.50
α = 0.25
α = 0.0
NOTE: CURVES BASED ON "SIMPLIFIED" METHOD OF ANALYSIS WITH $\gamma = 2\gamma'$ DEPTH RATIO, D/H INCREASED BY 20% TO EQUATE TO "CONVENTIONAL" METHOD.
$(2q_u - \gamma_e H)/\gamma' K_a H$
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0
DEPTH RATIO, D/H (INCREASE BY 20% TO 40% FOR S.F.)

Figure 6



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PROPOSED SHORING/SHEET PILE WALL AT POOL

Figure 7

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drilled caissons, 12 inches minimum in diameter, placed at a maximum spacing of 6 feet on center, and with a minimum embedment length of 7 feet below the bottom of the grade beam. 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. The design of the grade beam and caissons should be in accordance with the recommendations of the project structural engineer, and include the utilization of the following geotechnical parameters:

Creep Zone: 5-foot vertical zone below the slope face and projected upward parallel to the slope face.

Creep Load: The creep load projected on the area of the grade beam should be taken as an equivalent fluid approach, having a density of 60 pcf. For the caisson, it should be taken as a uniform 900 pounds per linear foot of caisson's depth, located above the creep zone.

Point of Fixity: Located a distance of 1.5 times the caisson's diameter, below the creep zone.

Passive Resistance: Passive earth pressure of 300 psf per foot of depth per foot of caisson diameter, to a maximum value of 4,500 psf may be used to determine caisson depth and spacing, provided that they meet or exceed the minimum requirements stated above. To determine the total lateral resistance, the contribution of the creep prone zone above the point of fixity, to passive resistance, should be disregarded.

Allowable Axial Capacity:

Shaft capacity : 350 psf applied below the point of fixity over the surface area of the shaft.

Tip capacity: 4,500 psf.

POOL RECOMMENDATIONS

The pool will be located near the lake shore behind the sheet pile wall and will be underlain by potentially liquefiable soils. Therefore, a drilled pier (pile) supported foundations is recommended (see Figure 7).

Pool Design Recommendations with Respect to Pools Underlain by liquefiable Soils

1. Due to the potential presence of liquefiable soils and lateral movement of the subgrade at the subject site, the equivalent fluid pressure to be used for the pool design should be 125 pcf for pool walls with level backfill. No sloped backfill condition is recommended within "D" from the edge of the pool, where "D" is the depth of the pool and shell. In addition, backdrains should be provided behind pool walls subjacent to any slopes. A sump may be necessary if backdrains cannot flow to an approved outlet via gravity. Sumps should be designed by the project civil engineer or architect and not allow the localized saturation of soils. This pressure assumes that expansive soils (E.I.>50) are not located any closer than 3 feet behind/below the pool shell.

2. Passive earth pressure may be computed as an equivalent fluid having a density of 200 pcf to a maximum lateral earth pressure of 2,000 psf.

3. An allowable coefficient of friction between soil and concrete of 0.30 may be used with the dead load forces.

4. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third. No friction component should be used with drilled pier caps.

5. Where pools are planned near structures (footings, landscape features, etc.), appropriate surcharge loads need to be incorporated into design and construction by the project design civil engineer.

6. The entire pool wall should be designed as "free standing" and be capable of supporting the water in the pool without soil support. The pool bottom (estimated to be a maximum of 8 feet below current grade plus the shell thickness) should be designed to withstand a potential slab movement of up to 1 to 2 inches, due to the expansive characteristics of the underlying bedrock and the anticipated varying depth of the pool.

7. The soil beneath the pool/spa bottom should consist of a 3-foot thick layer of very low to low expansive soils (E.I. = 0 to 50) that have been compacted to a minimum relative compaction of 90 percent to minimize damage due to expansive soil forces. If a fill/bedrock transition occurs beneath the pool bottom, as is likely, the bedrock should be overexcavated to a minimum depth of 3 feet, and be replaced as very low to low expansive (E.I. = 0 to 50) compacted fill. The lateral extent of the overexcavation should be 5 feet, minimally. Prior to placing fill within the overexcavation for the pool, a durable, non-permeable membrane should be placed on the overexcavation bottom. A horizontal blanket drain for leak detection should be provided.

8. Hydrostatic pressure relief valves should be incorporated into the pool and spa designs.

9. All fittings and pipe joints, particularly fittings in the side of the pool or spa, should be properly sealed to prevent water from leaking into the adjacent soils materials. Trenches and trench bottoms for pool/water feature improvements should be sloped to drain (in the event of a leak) away from the pool and existing and planned structures.

10. An elastic expansion joint (waterproof sealant) should be installed to prevent water from seeping into the soil at all deck joints.

11. A reinforced grade beam should be placed around the skimmer to provide support and mitigate cracking around the skimmer face.

12. In order to reduce unsightly cracking, deck slabs should be minimally reinforced with No. 3 reinforcing bars at 18 inches on center. All slab reinforcement should be supported to ensure proper mid-slab positioning during the placement of concrete.

13. Pool bottom or deck slabs should be founded entirely on a 3-foot thick layer of properly compacted very low to low expansive fill (E.I. = 0 to 50). Fill/bedrock transitions should be mitigated as specified above. Fill should be compacted to achieve a minimum 90 percent relative compaction. Prior to placement of concrete slabs, subgrade soils below the pool decking should be throughly watered to achieve a moisture content that is at least 10 percent above (1.1 times for very low to low expansive soils) the soil's optimum moisture content, to a depth of at least 3 feet below the bottom of slabs. This moisture content should be maintained in the subgrade soils during concrete placement to promote uniform curing of the concrete and minimize the development of unsightly shrinkage cracks.

14. In order to reduce unsightly cracking, the outer edges of pool decking to be bordered by landscaping, and the edges immediately adjacent to the pool/spa, should be underlain by 8-inch wide concrete cut-off walls (thickened edge) extending to a depth of at least 12 inches below the bottom of the slabs to prevent excessive infiltration of water under the pool deck. These thickened edges should be reinforced with two No. 4 bars, one at the top and one at the bottom. Deck slabs may be minimally reinforced with No. 3 reinforcing bars placed at 18 inches on center, in both directions. All slab reinforcement should be supported to ensure proper mid-slab positioning during the placement of concrete.

15. Surface and shrinkage cracking of the finish slab may be reduced or eliminated if a low slump and water-cement ratio are maintained during concrete placement. Excessive water added to concrete prior to placement is likely to cause shrinkage cracking.

16. Joint and sawcut locations for the pool deck should be determined by the design engineer and/or contractor. However, spacings should not exceed 10 feet.

17. It is imperative that adequate provisions for surface drainage are incorporated by the developer into their overall improvement scheme. Ponding water, ground saturation, and flows over slope faces are all conditions which must be avoided.

18. Soil generated from the pool, spa, and trench excavations to be used onsite should be compacted regardless of where it is to be placed. This material must not alter positive drainage patterns away from structural areas and toward the street or approved drainage devices.

19. Local irrigation and drainage should be diverted from all flatwork areas. Area drains and swales should be utilized to reduce the amount of subsurface water intrusion beneath the flatwork areas.

20. Bottoms of all pool utility trenches should be sloped away from the pool. Pump utility intersections should be evaluated by a mechanical consultant in light of the expansive soil conditions and the consequences of utility leakage.

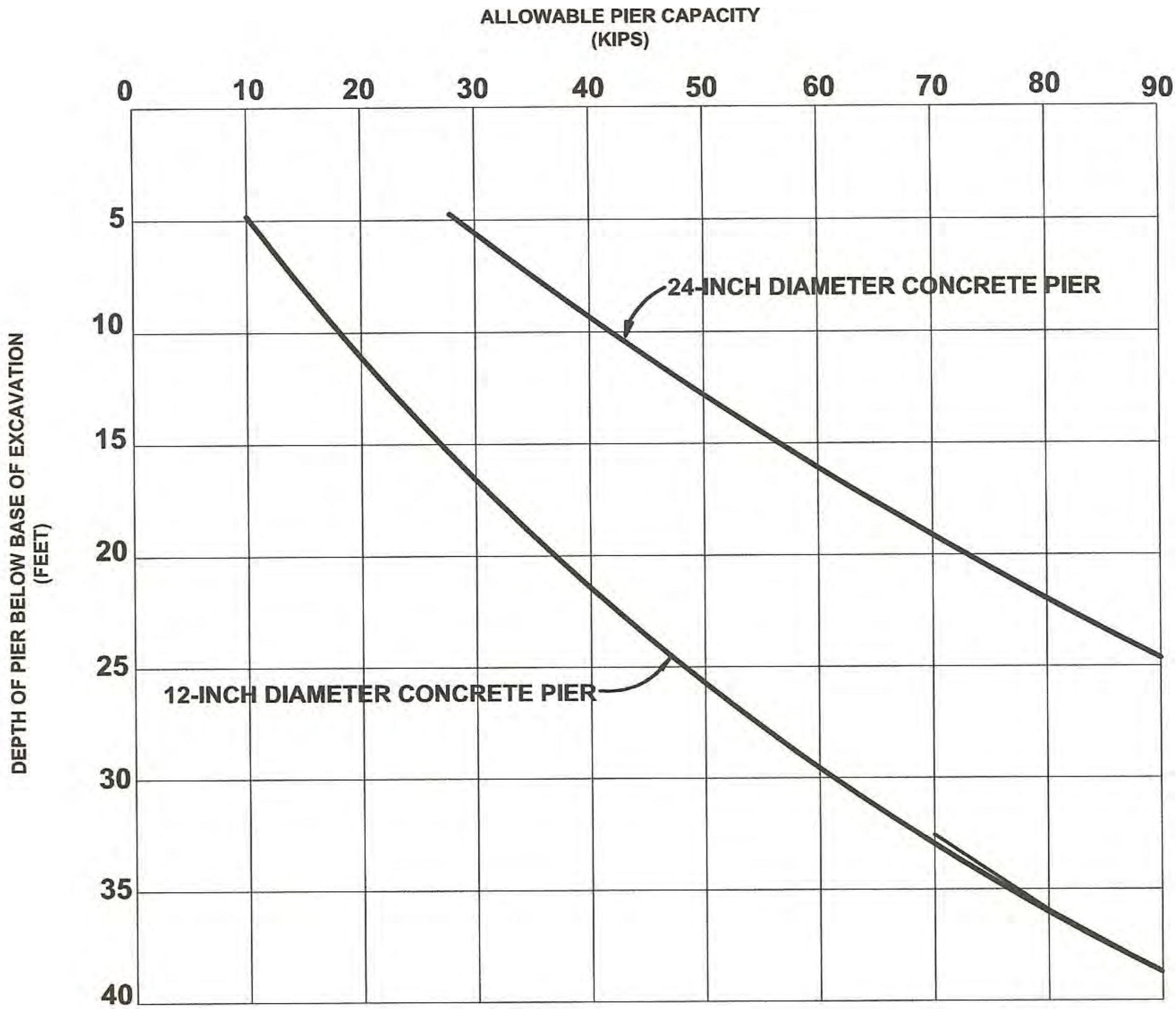
21. All pipe fittings/drains (pressure and gravity pipes), gas lines, electric lines to/from the pool/ponds/fountains should be designed for up to 8 inches of vertical or lateral deformation on a cyclic basis.

22. Free standing support posts (flags, slides, light poles, etc.) should be minimally supported on circular footings embedded 3 feet into select earth material and have a minimum diameter of 1 foot. The design skin friction should be 150 psf.

Drilled Pier and Grade Beam Foundation Recommendations - Pool

The proposed pool structure, underlain by compacted fill and potentially liquefiable sands should be supported by a drilled, cast-in-place, concrete pier and grade beam system. All drilled piers should extend a minimum of 10 feet into competent formational materials with tip elevations at approximately 1,220 to 1,227 feet MSL. Actual pier embedment should be finalized by the project's structural engineer based on the pier capacity chart (see Figure 8), and the structural capacity of the pier(s) used. The structural strength of the piers should be checked by the structural engineer or civil engineer specializing in structural analysis. Pier holes should be drilled straight and plumb. Locations (both plan and elevation) and plumbness should be the contractors responsibility.

The grade beam should be at a minimum of 24 inches by 24 inches in cross section and supported by drilled caissons 24 inches in diameter which are placed at a minimum spacing of 6 feet on center and supporting all structural columns. The design of the grade beam and caissons should be in accordance with the recommendations of the project structural engineer, and utilize the following geotechnical parameters:



1. **Minimum pier lengths should be 5 feet.**
2. **Capacities are allowable capacities (based on Factor of Safety = 2) and may be increased by one-third for short-term wind or seismic loads.**
3. **For uplift, use 75 percent of these capacities for single piers and 50 percent for piers in clusters.**

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PIER CAPACITY CHART

Figure 8

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Passive Resistance

Passive earth pressure of 500 pcf per foot of caisson depth, to a maximum value of 4,000 psf may be used to determine pier depth and spacing, provided that they meet or exceed the minimum requirements stated above. No friction component should be used with drilled pier caps or interconnected grade beams.

Point of Fixity

The point of fixity should be located at a distance equivalent to one-third of the caissons length below the bottom of the grade beam.

Allowable Axial Capacity

A shaft capacity of 400 psf should be applied over the surface area of the shaft located in bedrock only. The tip bearing capacity should be limited to 6,000 psf.

Caisson Construction

1. The excavation and installation of the drilled caissons should be observed and documented by the project geotechnical engineer to verify the recommended depth.

2. The drilled holes should be cased, specifically below the water table to prevent caving. The bottom of the casing should be at least 4 feet below the top of the concrete as the concrete is poured and the casing is withdrawn. Dewatering may be required for concrete placement if significant seepage or groundwater is encountered during construction. This should be considered during project planning. The bottom of the drilled caisson should be cleared of any loose or soft soils before concrete placement.

3. The exact depths of caissons should be determined during the final precise grading plan review.

4. Proper slump underwater type concrete (Type V) should be used and should be delivered through tremie pipe. We recommend that concrete be placed through the tremie pipe immediately subsequent to approved excavation and steel placement. Care should be taken to prevent striking the walls of the excavations with the tremie pipe during concrete placement.

5. All footing excavations should be observed and approved by the geotechnical consultant prior to placement of concrete forms and reinforcement.

6. Drilled pier steel reinforcement cages should have spacers to allow for a minimum spacing of steel from the side of the pier excavation. All reenforcing bars should be

epoxy coated due to the corrosive environment. The need for epoxy coated steel in below grade walls and grade beams should be evaluated by a structural consultant.

7. During pier placement, concrete should not be allowed to free fall more than 5 feet.

8. Concrete used in the foundation should be tested by a qualified materials testing consultant for strength and mix design.

9. Concrete floor slabs, in association with Alternative B, should be structural slabs and shall be in accordance with Table 19-A-2 of the UBC (ICBO, 1997), for "concrete intended to have a low permeability when exposed to water," (i.e., a maximum water-cement ratio of 0.50 and a minimum strength of 4,000 psi), to mitigate the effects from post-development perched water and to impede water vapor transmission. Slabs should be able to span between piers without the support of subgrade soils. Slab underlayment should consist of 2 inches of washed sand (as specified previously) placed above a vapor retarder consisting of 15-mil polyvinyl chloride, or equivalent, with all laps sealed per UBC (ICBO, 1997). The vapor retarder shall be underlain by 4 inches of pea gravel placed directly on the slab subgrade, and should be sealed to provide a continuous water-proof retarder under the entire slab, as discussed above. All slabs should be additionally sealed with suitable slab sealant. Nuisance cracking may be lessened by the addition of engineered reinforcing fibers in the concrete and careful control of water/cement ratios.

Drilled Pier and Grade Beam Foundation Settlement

Drilled pier and grade beam foundations should be designed to accommodate ½ inch over a 40-foot horizontal span.

Corrosion and Concrete Mix

Upon completion of grading, laboratory testing should be performed of site materials for corrosion to concrete and corrosion to steel. Additional comments may be obtained from a qualified corrosion engineer at that time. At a minimum, the corrosion consultant should utilize the preliminary corrosion testing in the report.

DRIVEWAY, FLATWORK, AND OTHER IMPROVEMENTS

The 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 any homeowners or homeowners association 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 to about 12 inches below subgrade 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. If very low expansive soils are present, the rock or gravel or sand may be deleted. 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. Driveways, sidewalks, and patio slabs adjacent to the house should be separated from the house 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 house.

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 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. It should be kept in mind that drainage reversals could occur, including post-construction settlement, if relatively flat yard drainage gradients are not periodically maintained by the homeowner or homeowners association.

12. Air conditioning (A/C) units 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.

SHORE PROTECTION DESIGN CONSIDERATIONS

The proposed project includes a lake front retaining wall which, when the lake levels are high enough, will be subject to small wind waves, boat wakes and varying water levels. The key design variables for the shore protection retaining wall, aside from the site soil strength parameters and ground water level, are the lake water levels, the boat wake size, the wind wave size, and the maximum expected scour depth at the base of the wall. These variables will be discussed separately and then combined together to determine the most conservative design scenario for the shore protection wall.

Water Levels

Water levels can vary greatly within Lake Elsinore. According to the client, the maximum lake level is at about elevation ±1,267 feet MSL. The lake is currently at about elevation ±1,256 feet MSL, but has been as low or lower that ±1,243 feet MSL in the past few years. The maximum lake level is controlled by the spill way in the southeast corner of the lake. This spill way is at about elevation ±1,266 feet MSL and is located beneath the West Limited Street bridge. The base of the proposed retaining wall is at about elevation 1,253 feet MSL with decorative stones and landscaping at the toe of the wall extending down to about elevation 1,250 feet MSL. Lake levels below the base of the wall at about elevation 1,253 feet MSL will not impact the wall from the lake side but may impact ground water elevations behind the wall.

Boat Wakes

Boat wakes within Lake Elsinore can reach up to 3 feet in height depending upon the vessel propulsion and hull shape. These wakes are typically generated by boats traveling in the deeper portions of the lake and not near the shoreline. The shoreline profile gradients are relatively flat at about 10:1 (h:v). As the wakes travel away from the vessel, into shallower and shallower water, the wake height is reduced. Boat wakes occur only during the passing of a boat and are characterized as transient in nature. Boat wakes approaching the proposed shore protection wall will be dissipated and reflected for the most part by the decorative stones and landscaping at the toe of the wall. Due the transient nature of boat wakes, as compared to persistent, similar height, wind waves, they are not considered in the design of the shore protection.

Wind Waves

Waves can be generated within Lake Elsinore by local winds. These waves will approach the shoreline and impact on the proposed shore protection. Using the shallow water wind wave generation models detailed in the US Army Corps of Engineers Shore Protection Manual and Coastal Construction Manual (USACOE, 1984 and 2004) design wave heights and periods can be determined. Lake Elsinore is an essentially rectangular lake. As measured from the east, the maximum fetch (distance over the water) to the proposed shore protection is about ±8,700 feet. As measured from the south east, diagonally, the maximum fetch to the proposed shore protection is about ±13,600 feet. Wave heights for each fetch were calculated using a wind speed of about 50 miles per hour for a duration of 3 hours. The predicted shallow water wave height from the east is 2.83 feet with a period of 2.7 seconds. The predicted shallow water wave height from the south east is 3.5 feet with a period of 3.1 seconds. For design purposes the larger wave height of 3.5 feet and period of 3.1 seconds will be used. This height is greater and therefore more conservative than the boat wake height discussed above. Linear wave theory was used to determine the peak horizontal water velocity under the design wave. The design peak horizontal water velocity is 5 fps.

Maximum Scour

Wave/wake action at the toe of the proposed, shore protection structure can scour away the native sandy soils. As currently proposed, the base of the proposed wall is protected by decorative stones and landscaping. If these stones and landscaping are properly designed, they will not scour under the action of waves and wakes. The minimum size for the decorative stones, that will prevent scouring, are discussed below. If the base of the wall is not protected by stones and landscaping, the maximum scour can be as much as three feet below the finished grade at the base of the wall. This scour would occur when large (approximately 3 feet in height) wind waves hit the wall, when the lake level is above elevation 1,253 feet MSL. For design purposes, with no toe protection, the maximum scour at the shore protection is about elevation $\pm$1,250 feet MSL. With the toe protection in place the maximum scour on the lake side of the wall will be the elevation where the toe protection counters the earth forces on back side of the wall. This elevation will need to be determined by the design civil engineer, but will likely range from elevation 1,253 feet MSL to elevation 1,260 feet MSL.

Toe Protection

The weight of the stone for the wall toe protection is determined using the Caltrans rock slope protections (RSP) design method as detailed in State of California, Department of Transportation (2000). The minimum toe stone size for the design velocity of 5 fps and the slope of the decorative rock at 2:1 (h:v) is 125 pounds. The toe stone needs to be a minimum of 125 pounds and at least two layers thick to prevent scouring of the toe protection by waves. Larger stone may be required if the toe is to resist the earth forces on the landward side of the wall.

Wave Runup

With the lake level at its maximum elevation 1,266 feet MSL and wind waves on the order of 3 feet in height, it is possible that wave runup will overtop the proposed shore protection at elevation 1,267 feet MSL. This can occur when there is toe protection or when there is no toe protection. Wave runup is defined as the vertical height above the still water level to which a wave will rise on a structure (wall or toe protection) of infinite height (see Figure 9). Overtopping is the flow rate of water over the top of a finite height structure (the wall) as a result of wave runup. In order to determine the potential for wave runup to impact the site improvements, the wave runup and overtopping for the shore protection is calculated using the US Army Corps of Engineers Automated Coastal Engineering System (ACES). ACES is an interactive computer based design and analysis system in the field of coastal engineering. The methods to calculate runup and overtopping implemented within this ACES application are discussed in greater detail in Chapter 7 of the Shore Protection Manual (1984).

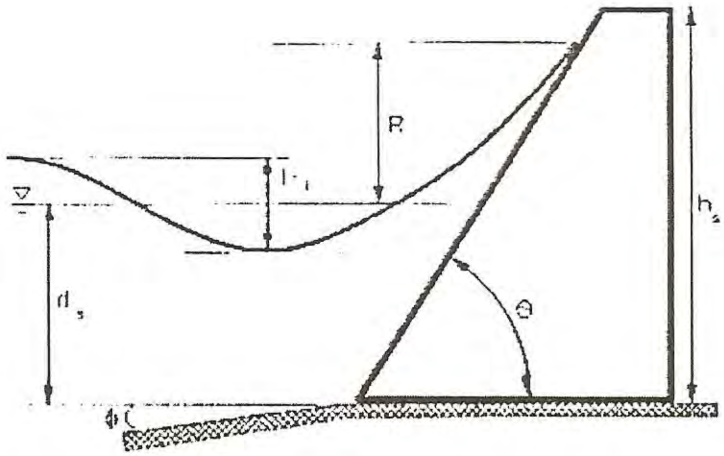


Figure 9. Wave runup terms from ACES manual.

For a vertical wall top at elevation ±1,267 feet MSL, with the lake level at elevation 1,266 feet MSL, and 3.5-foot waves with a 3.1-second period, the wall can be overtopped at a rate of about 6 inches to 1 foot of water per wave per foot of wall (4.0 ft^3/s-ft). For the case with the sloping toe protection, the top of wall can be overtopped at a rate of about 2.2 ft^3/s-ft. If the height of the top of wall is extended to elevation 1,269 feet MSL then no overtopping is calculated. For top of shore protection below elevation 1,269 feet MSL consideration should be given to the impact of water coming over the top of the wall to the improvements behind the wall. Site drainage should be configured to allow for these waters to flow back into the lake. The velocity of the overtopping water is not sufficient to cause structural damage to properly design improvements, but is sufficient to carry away lounge chairs, tables, and other light weight improvements.

Shore Protection Wall Discussion

The following discussion is provided to help with the preliminary dimensioning for the proposed shore protection. The design parameters used herein are intended only for discussion and preliminary dimensioning. The final design will need to be performed by the project design civil engineer and will be subject to our review for compliance with the geotechnical and shore protection design parameters contained herein.

It is our understanding that either a sheet pile wall or a gravity type wall are being considered. Our discussion herein will be for the sheet pile type wall, with and without toe protection. The gravity type wall will need to withstand the earth forces on the back side. The wave forces on the front side of the wall are much smaller than the earth forces on the back side, including lateral seismic induced forces, and need not be considered in the design of either type wall.

A typical sheet section for shoreline protection is the Hoesch H-1700K. The H-1700K has a section modulus of 31.62 cubic inches, a width of 22.64 inches, and a depth of 13.78 inches. The design input for the sheet pile wall is; Top of wall (T.O.W.) at 1,267 feet MSL, toe at 1,250 feet MSL, and a 2,000 psf surcharge to represent lateral seismic loads. For the case of no toe protection the wall will need to have a tie back system in order to keep the sheet total length to about 30 feet and top of wall deflection to less than 1 inch. For the case of toe protection (resistance) to elevation 1,263 feet MSL the sheet length can be reduced to about 20 feet and no tie back system is necessary.

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 each homeowner and/or any homeowners association.

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 each homeowner. 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. Consideration should be given to avoiding construction of planters adjacent to structures (buildings, pools, spas, 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.

Toe of Slope Drains/Toe Drains

Where significant slopes intersect pad areas, surface drainage down the slope allows for some seepage into the subsurface materials, sometimes creating conditions causing or contributing to perched and/or ponded water. Toe of slope/toe drains may be beneficial in the mitigation of this condition due to surface drainage. The general criteria to be utilized by the design engineer for evaluating the need for this type of drain is as follows:

- Is there a source of irrigation above or on the slope that could contribute to saturation of soil at the base of the slope?
- Are the slopes hard rock and/or impermeable, or relatively permeable, or; do the slopes already have or are they proposed to have subdrains (i.e., stabilization fills, etc.)?
- Are there cut-fill transitions (i.e., fill over bedrock), within the slope?
- Was the lot at the base of the slope overexcavated or is it proposed to be overexcavated? Overexcavated lots located at the base of a slope could accumulate subsurface water along the base of the fill cap.
- Are the slopes north facing? North facing slopes tend to receive less sunlight (less evaporation) relative to south facing slopes and are more exposed to the currently prevailing seasonal storm tracks.
- What is the slope height? It has been our experience that slopes with heights in excess of approximately 10 feet tend to have more problems due to storm runoff and irrigation than slopes of a lesser height.
- Do the slopes "toe out" into a residential lot or a lot where perched or ponded water may adversely impact its proposed use?

Based on these general criteria, the construction of toe drains may be considered by the design engineer along the toe of slopes, or at retaining walls in slopes, descending to the rear of such lots. Following are Detail 4 (Schematic Toe Drain Detail) and Detail 5 (Subdrain Along Retaining Wall Detail). Other drains may be warranted due to unforeseen conditions, homeowner irrigation, or other circumstances. Where drains are constructed during grading, including subdrains, the locations/elevations of such drains should be surveyed, and recorded on the final as-built grading plans by the design engineer. It is recommended that the above be disclosed to all interested parties, including homeowners and any homeowners association.

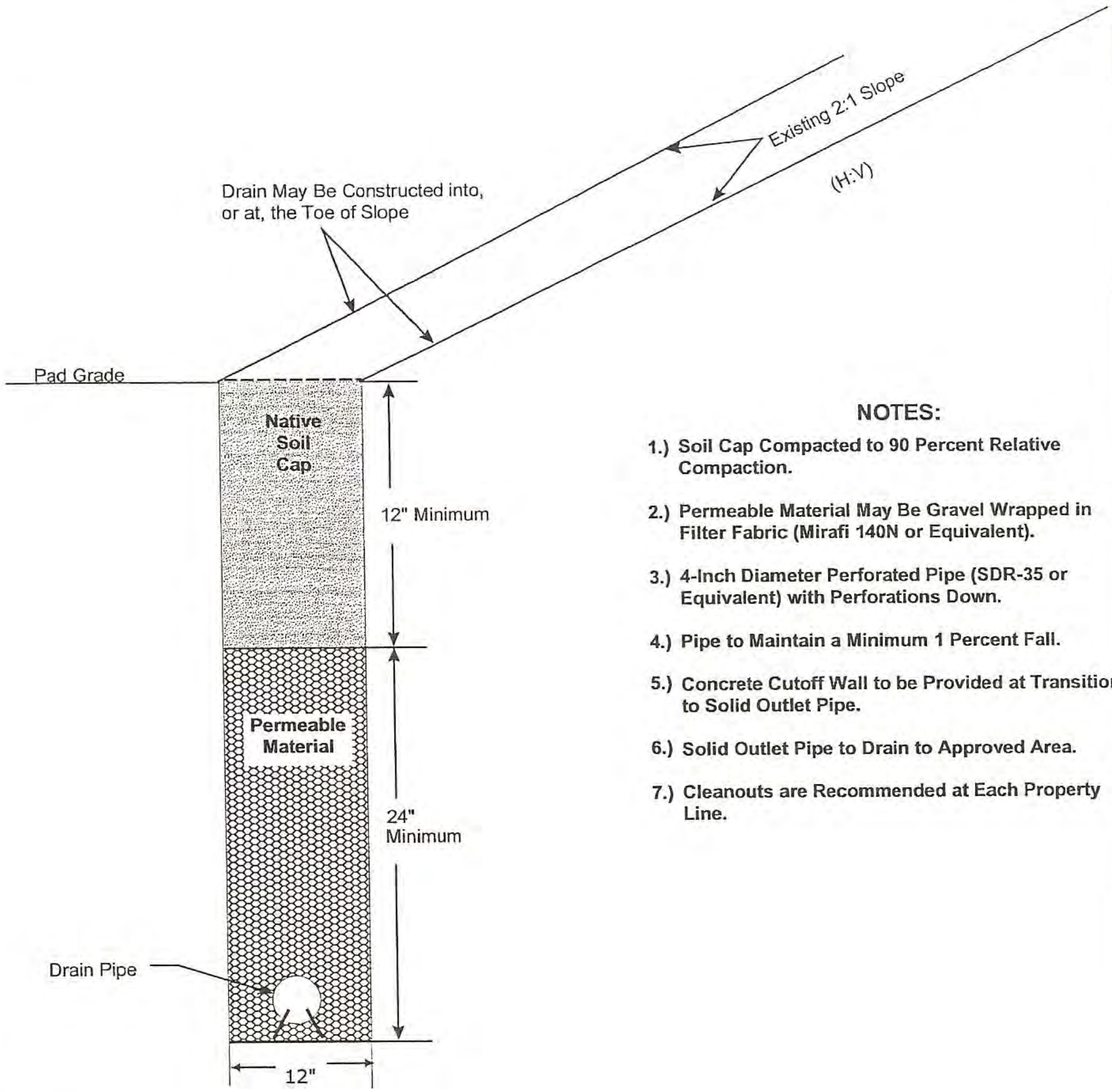
Erosion Control

Cut and fill slopes will be subject to surficial erosion during and after grading. Onsite earth materials have a moderate to 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.

DETAILS

N . T . S .

SCHEMATIC TOE DRAIN DETAIL



NOTES:

1.) Soil Cap Compacted to 90 Percent Relative Compaction.

2.) Permeable Material May Be Gravel Wrapped in Filter Fabric (Mirafi 140N or Equivalent).

3.) 4-Inch Diameter Perforated Pipe (SDR-35 or Equivalent) with Perforations Down.

4.) Pipe to Maintain a Minimum 1 Percent Fall.

5.) Concrete Cutoff Wall to be Provided at Transition to Solid Outlet Pipe.

6.) Solid Outlet Pipe to Drain to Approved Area.

7.) Cleanouts are Recommended at Each Property Line.

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SCHEMATIC TOE DRAIN DETAIL

DETAIL 4

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DETAILS

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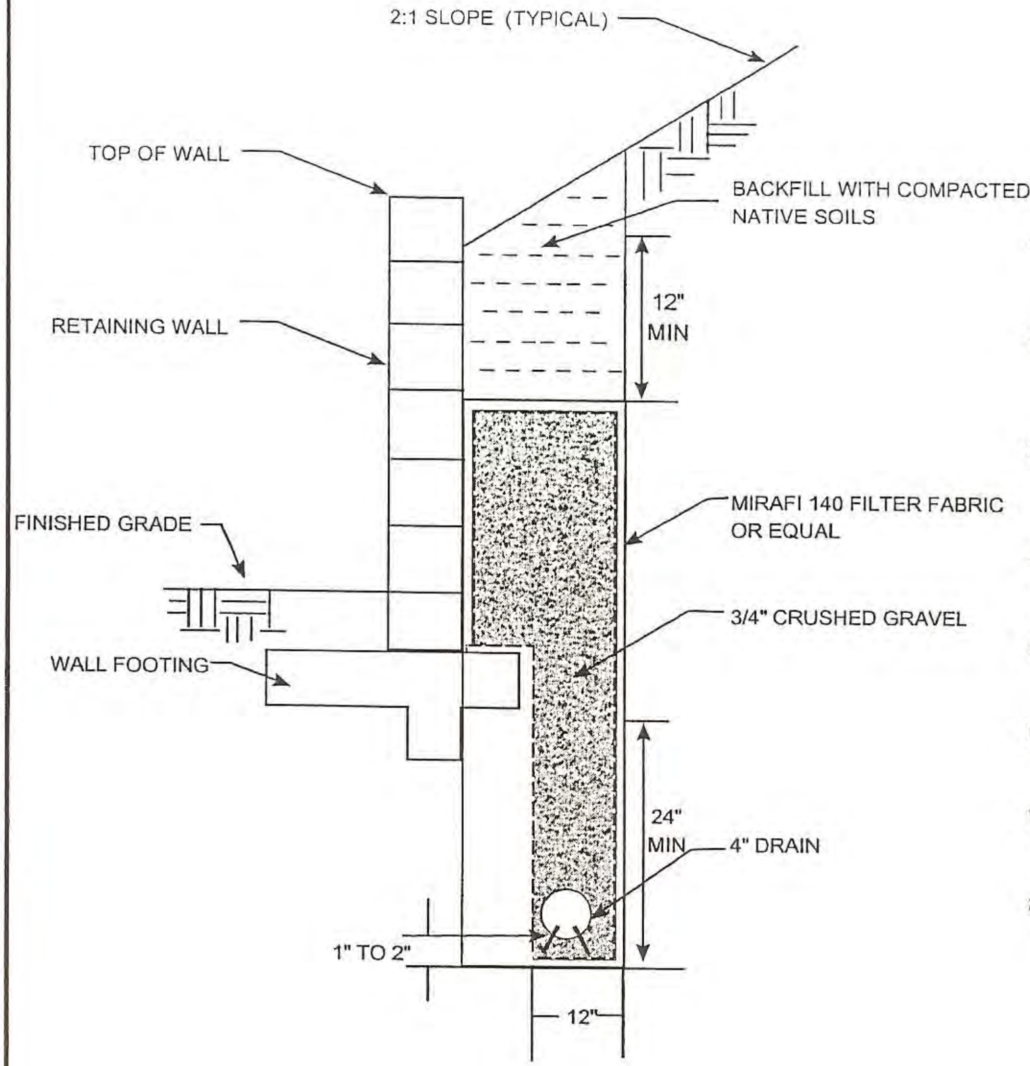

NOTES:

1.) Soil Cap Compacted to 90 Percent Relative Compaction.

2.) Permeable Material May Be Gravel Wrapped in Filter Fabric (Mirafi 140N or Equivalent).

3.) 4-Inch Diameter Perforated Pipe (SDR-35 or Equivalent) with Perforations Down.

4.) Pipe to Maintain a Minimum 1 Percent Fall.

5.) Concrete Cutoff Wall to be Provided at Transition to Solid Outlet Pipe.

6.) Solid Outlet Pipe to Drain to Approved Area.

7.) Cleanouts are Recommended at Each Property Line.

8.) Compacted Effort Should Be Applied to Drain Rock.

SUBDRAIN ALONG RETAINING WALL DETAIL

NOT TO SCALE

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SUBDRAIN ALONG RETAINING WALL DETAIL

DETAIL 5

Geotechnical • Coastal • Geologic • Environmental

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 house, 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 (e.g., pools, spas, etc.) are planned for the site, recommendations concerning the geological or geotechnical aspects of design and construction of said improvements could be provided upon request. Pools and/or spas should not be constructed without specific design and construction recommendations from GSI, and this construction recommendation should be provided to the homeowners, any homeowners association, and/or other 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 homeowners, 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.

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 homeowner improvements, such as flatwork, spas, pools, 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 to homeowners/homeowners associations for geotechnical aspects, including irrigation practices, the conditions outlined above, etc., prior to any sales. At that stage, GSI will provide homeowners 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. Such recommendations may include a thicker slab, thicker underlayment, 4,000 psi concrete with a water-cement ratio of 0.5, etc. Homeowners should be advised of the potential for water or 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.