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File No. WE19-020899

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c/o Wade Lewis
RWC, LLC
P.O. Box 1025
Santa Paula, CA 93061

SUBJECT: **Geotechnical Engineering Investigation Report:** Proposed Multi-Family Residential Mixed-Use Development, 4209 Carpinteria Avenue, City of Carpinteria.

Dear Mr. Lewis:

This report presents the findings from our geotechnical engineering investigation for the proposed construction of a multi-family residential mixed-use development at the subject property. The purpose of this investigation was to determine and evaluate the nature, distribution, and engineering properties of the earth materials at the site, so that we may provide suitable recommendations for site preparations, and foundation design criteria for the planned site improvements at the subject property.

SCOPE OF WORK

The scope of work for this investigation included the completion of the following tasks:

1) Review of available geologic data pertaining to the site and its vicinity, including the following:

   a) California Division of Mines and Geology Open-File Report 76-5LA, Weber, 1975.

   b) California Division of Mines and Geology Preliminary Report 14, 1973.

   c) Geologic Map of the Carpinteria Quadrangle, 1986, T .W. Dibblee, Jr.

2) Review of Site Plan showing the planned site improvements provided by *RWC, LLC*.

3) Logging and sampling of the earth materials exposed by one 8-inch diameter hollow stem auger boring and seven manually excavated test pits within the building areas for logging, sampling, soil classification, groundwater evaluation, and liquefaction hazard evaluation.

4) Laboratory testing on the retrieved samples of the earth materials to determine pertinent engineering properties for project design purposes.

5) Geotechnical engineering analysis of the field and laboratory data with respect to the proposed new site development.

6) Preparation of this report to present our findings and recommendation for site preparations, grading, pavement design, and foundation design criteria.

A Plot Plan showing the approximate location of the exploratory boring is included as Plate 1, attached at the end of this report. Descriptions of the earth materials encountered in the exploratory boring are provided on the enclosed and Boring Log, Plate 2.1 and Test Pit Log, Plate 3.1.

PROPOSED DEVELOPMENT

The findings and recommendations contained in this report are based on information provided to this office by the client. Based on information provided to this office, the proposed site improvements will consist of the construction of the multi-family residential mixed-use development at the approximate location shown on the Plot Plan, Plate 1. It is our understanding that the proposed development will consist of a commercial building, a covered parking structure, and a residential building with at-grade paved parking. It is our understanding that the structures will be constructed utilizing conventional foundation systems and concrete slabs-on-grade at or near the existing grade, so that no significant grading or retaining wall is required. Site preparation work associated with the planned new construction is anticipated to include the demolition of the existing structures, removal and recompaction of the near surface soils to establish suitable soils conditions for support of the planned structures and concrete slabs, and fine grading as necessary to establish positive drainage control.

FINDINGS

Site Description

The property is located at the southern side of Carpinteria Avenue within the City of Carpinteria (see Site Location Map, Figure 1). The building site is situated within flat alluvial terrain and is bordered by a residential lot to the east, railroad tracks to the south, Estero Street to the west, and Carpinteria Avenue to the north. The property is being utilized as a storage yard and contains a storage trailer, storage containers, and parked vehicles that will be removed to accommodate the proposed site development.

Drainage

Drainage is by sheetflow runoff. The proposed building area is not located in an area that is subject to concentrated flows or flooding.

SUBSURFACE CONDITIONS

Based on our site investigation and review of the "Geologic Map of the Carpinteria Quadrangle" by T.W. Dibblee, Jr., the site is underlain by artificial fill, in-turn underlain by artificial fill, underlain by native soil, in-turn underlain alluvium. Descriptions of the earth materials encountered are provided in the Boring Log, included as Plate 2.1 and Test Pit Log, Plate 3.1.

Artificial Fill (Af)

Artificial fill was encountered to a variable depth of approximately 1.5 to 4 feet, as measured from below the existing grade, in our exploratory excavations. The artificial fill consists of medium brown silty sand with gravel to dark brown clayey silty sand. The artificial fill was found to be slightly moist to moist and poorly to moderately compacted and is not considered to be suitable as a base for support of the planned structural loads and areas to receive concrete slabs-on-grade, paving, or engineered fill.

Native Soil (Ns)

The artificial fill was found to be underlain by native soil to a variable depth of approximately 3.5 to 5 feet, as measured from below the existing grade, in our exploratory excavations. The native soil consists of brown to dark reddish brown silty sand. The native soil was found to be locally porous, moist to wet, and slightly dense and is not considered to be suitable as a base for support of the planned structural loads and areas to receive concrete slabs-on-grade, paving, or engineered fill.

Alluvium (Qa)

The artificial fill and native soil were found to be underlain by alluvium to the depth of exploration of approximately 50 feet. The alluvium is comprised of lenses of layers of dark brown to greyish brown fine-grained silty sand, clayey silty sand, and sand. The alluvium below a depth of approximately 5 feet was found to be dense and saturated and will provide a suitable base for support of engineered fill and the planned structural loads. Due to the potential for high groundwater and saturated soils conditions, removal excavation bottoms may need to be stabilized prior to placing engineered fill.

Groundwater

Free ground water was encountered at a depth of approximately 6 feet and stabilized at a depth of approximately 5 feet. Groundwater elevations are dependent on seasonal precipitation, irrigation, land use, climatic conditions, among other factors, and as a result, fluctuates. Therefore, water levels at the time of construction and during the life of the facility may vary from the observations or conditions at the time of our field exploration.

FAULTING AND SEISMICITY

The intensity of ground shaking during an earthquake can result in a number of phenomena classified as ground failure, which include ground rupture due to faulting, landslides, liquefaction, lurching, and seismically induced settlement. Other seismic hazards include Seiches and tsunamis. Descriptions of each of these phenomenon and an assessment of each, as it affects the proposed site, are included in the following paragraphs. The Seismic Hazards Mapping Act of 1990, which became effective in 1991, requires mitigation of seismic hazards to a level that does not cause collapse of the building intended for human occupancy, but it does not require mitigation to a level of no ground failure or structural damage.

Seismicity Study

Based on the USGS Unified Hazard Tool computer program, the computed site peak ground acceleration and magnitude for a 50-year exposure and 2% exceedance is 1.05 g and 7.11, respectfully.

CBC Seismic Design Parameters

Seismic design coefficients and parameters for the project sites have been determined utilizing the "OSHPD Seismic Design Maps" developed by the Structural Engineers Association of California (SEA). The program incorporates seismic provisions set forth in the 2016 California Building Code (CBC) and ASCE 7-16 procedures. The following seismic parameters may be utilized for structural design.

Site Classification	Spectral Response Accelerations		Site Seismic Coefficients	
	SMs	**SM1**	**Fa**	**Fv**
D	**2.266**	**1.093**	**1.0**	**1.7**

Faulting

Southern California is a tectonically active region subject to hazards associated with earthquakes and faulting. Faults are classified as either active, potentially active, or inactive. Active faults are defined by the State of California as a fault that has exhibited surface displacement within the last 11,000 years. Potentially active faults are defined by the State of California as those with a history of movement between 11,000 and 1.6 million years. Alquist-Priolo Earthquake Fault Zones are zones that have been established by the State as areas that contain active faults, and projects that are located within these zones require that a fault investigation be performed to determine if active faulting affects the site. The site is *not* located in an Alquist-Priolo Earthquake Fault Zone (CDMG 1998).

Shallow Ground Rupture

Ground surface rupture occurs when movement along a fault is sufficient to cause a gap or rupture where the upper edge of the fault zone intersects the ground surface. Ground rupture associated with faulting is typically characterized by relatively short segments of faulting that occur over a broad area of the upper plate. In some cases, particularly in unconsolidated alluvial sediments, *secondary ground ruptures* can develop from a number of causes not necessarily related directly to surface rupture of the causative fault. The secondary processes may include ground shaking, seismic settlement, landslides, and liquefaction. Since there are no known active or potentially active faults passing through the site, the potential of on-site ground rupture due to movement on an underlying fault in not considered a significant hazard. The potential for ground rupture due to other causes is discussed in the following paragraphs.

Earthquake-Induced Landsliding

Landslides are slope failures that occur where the horizontal seismic forces act to induce slope failure. As the site is relatively flat, on-site earthquake-induced landsliding is not considered to be a hazard.

Ground Lurching

Ground lurching is defined as seismic motion at right angles to a cliff or bluff, or more commonly to a stream bank or artificial embankment that results in failure of the slope or bluff in the direction to which it is unsupported. The initial effect is to produce a series of more or less parallel cracks separating the ground into rough blocks. These cracks are generally parallel with the top of the slope or embankment. The topography of the site in the vicinity of the proposed improvements does *not* lend itself to this type of lurching.

Lurching is also sometimes used to describe undulating surface waves in the soil that have some similarities to the ground oscillation mentioned below in the *Liquefaction* section, but generally occurs in soft, saturated, fine-grained soils during seismic excitation. When this phenomenon occurs adjacent to bodies of water, lurching can continue for a short time after the seismic shaking stops. The soil conditions at this site are *not* typical of those associated with lurching, and we do *not* consider this type of lurching to be a risk at this site.

Seiches and Tsunamis

Seiches are an oscillation of the surface of an inland body of water that varies in period from a few minutes to several hours. Seismic excitations can induce such oscillations. Tsunamis are large sea waves produced by submarine earthquakes or volcanic eruptions. Since the site is *not* located close to an inland body of water and is located outside the zone of a tsunami runup, the risk of these two hazards is *not* pertinent to this site.

Liquefaction

The shear strength of soils is governed by effective stresses, which are equal to the total stresses minus the pore water pressures. In saturated, cohesionless soils, such as sands, pore water pressures tend to increase with cyclic loading, such as that caused by earthquakes. Liquefaction describes phenomena in which cyclic stresses produced by ground shaking induce excess pore water pressures in cohesionless soils that are about equal to the total stresses, resulting in near zero shear strength in the soil, causing the soil behaves as a viscous fluid. Liquefied soils may thereby acquire a high degree of mobility leading to damaging deformations. Liquefaction susceptibility under a given earthquake is related to the gradation and relative density characteristics of the soil, the in-situ stresses prior to ground motion, and the depth to the water table, as well as other factors.

As a general rule, sites susceptible to liquefaction in seismically active areas (a) contain cohesionless soils and fine-grained soils with less than 15% of clay sized particles, with liquid limits less than 35, and moisture contents greater than 90% of the liquid limit, (b) have an anticipated high groundwater level, including perched conditions, within 50 feet of the surface, (c) contain soils with a relative density less than about 70%, (d), contain younger deposits that are more susceptible to liquefaction than older deposits, and (e) have a plasticity index of less than 18.

Liquefaction related or liquefaction-induced phenomena include *lateral spreading*, *ground oscillation*, *flow failure*, *reduction of bearing strength*, *ground fissuring*, and *sand boils*. *Lateral spreading* is the lateral movement of stiff, surficial blocks of sediments as a result of a subsurface layer liquefying. The lateral movements can cause ground fissures or extensional, open cracks at the surface as the blocks move toward a slope face, such as a stream bank or in the direction of a gentle slope. When the shaking stops, these isolated blocks of sediments come to rest in a place different from their original location and may be tilted.

Ground oscillation occurs when liquefaction occurs at depth but the slopes are too gentle to permit lateral displacement. In this case, individual blocks may separate and oscillate on a liquefied layer. Sand boils and fissures are often associated with this phenomenon.

Flow failure, a more catastrophic mode of ground failure than either lateral spreading or ground oscillation, involves large masses of liquefied sediment or blocks of intact material riding on a liquefied layer moving at high speeds over large distances. Generally flow failures are associated with ground slopes steeper than those associated with either lateral spreading or ground oscillation.

Bearing strength decreases with a decrease in effective stress. *Loss of bearing strength* occurs when the effective stresses are reduced due to the cyclic loading caused by an earthquake. Even if the soil does not liquefy, the bearing of the soil may be reduced below its value either prior to or after the earthquake. If the bearing strength is sufficiently reduced, structures supported on the sediments can settle, tilt, or even float upward in the case of lightly loaded structures such as gas pipelines.

Ground fissuring and *sand boils* are surface manifestations associated with liquefaction and lateral spreading, ground oscillation, and flow failure. As apparent from the above descriptions, the likelihood of ground fissures developing is high when lateral spreading, ground oscillations, and flow failure occur. Sand boils occur when the high pore water pressures are relieved by drainage to the surface along weak spots that may have been created by fissuring. As the water flows to the surface it can carry sediments, and if the pore water pressures are high enough they can create a gusher (sand boil) at the point of exit.

Evaluation of Liquefaction Potential

The site is located in an area considered being susceptible to hazards associated with liquefaction. The results of our field exploration and laboratory testing programs also indicate that the subject site meets all the above-mentioned conditions for being susceptible to liquefaction. We performed a liquefaction analysis to further evaluate the potential and extent of possible liquefaction at this site. The results of this analysis along with other geotechnical information about the area were then used to evaluate the different liquefaction-induced phenomena mentioned herein.

Exploratory Boring B-1 was excavated to a depth of approximately 50 feet to assess the liquefaction hazard potential at the site. The geotechnical data obtained from the boring and our laboratory test results, including standard penetration test data (SPT), percent fines, and clay fraction, were utilized in our evaluation of liquefaction hazard potential at the site. Alluvial soils consisting of lenses and layers of clayey silty sand, silty sand, and sand were encountered to the depth of exploration of 50 feet.

At the time of our field exploration, groundwater was encountered at a depth of 6 feet and stabilized at a depth of 5 feet in exploratory boring B-1. The liquefaction hazard and settlement analyses were performed utilizing a groundwater level of 5 feet.

We performed a liquefaction analysis with LiquefyPro (V.5) by CivilTech Corporation using the geotechnical data obtained from the boring and our laboratory test results to further evaluate the potential and extent of possible liquefaction at this site. The method following the recommendations of Tokimatsu and Seed, and Idris and Seed was used with the design-level seismic event (earthquake magnitude of 7.11, the weighted magnitude used to generate seismic risk, and a site acceleration of 1.05g, the computed site peak acceleration for a 50-year exposure and 2% exceedence) to perform the liquefaction evaluation. Blow counts used for the liquefaction evaluation were based on the blow counts from the California sampler and the Standard Penetration sampler. Blow counts using a modified California sampler are adjusted to equivalent blows of a standard penetration test sampler. A multiplier of 2/3 was used to convert blows from the California sampler to an equivalent SPT value, although in clayey soils a slightly higher value could be justified. The measured blow counts were further adjusted for borehole diameter, rod length, sampling method and delivered energy to correspond to a driving-energy level of 60% (N_{60}).

The results of our liquefaction analysis are shown on the computer generated work sheets in Appendix II. The analysis indicates that several soil layers may be subject to liquefaction during the design earthquake event. Based on the laboratory and field testing, the site is underlain by several layers of soil having a N-60 SPT blow counts exceeding 30, so that liquefaction hazard potential is unlikely within these materials (Bray and Sancio, 2006). Although, classic cyclic liquefaction is unlikely to occur within these soils, strength loss may occur due to earthquake induced monotonic shearing. Since the soils at the site may liquefy, further analyses were performed to evaluate the potential and extent of *lateral spreading*, *ground oscillation*, *flow failure*, *reduction of bearing strength*, and surface manifestations of *sand boils* and *ground fissuring*.

Lateral Spreading Due to Liquefaction

An evaluation of lateral spreading was made with the procedure described by Bartlett and Youd (2002). The results of this analysis show that lateral spreading is unlikely, since the adjusted sampler blow counts ($N_1|_{60}$) in the liquefiable soils exceed 15 and due to the absence of free faces within close proximity to the site.

Ground Surface Manifestation Due to Liquefaction

An evaluation of the potential for ground damage due to the occurrence of ground fissuring, ground oscillation, and sand boils was made using the procedure of Ishihara (1985). This procedure is only valid for sites *not* susceptible to lateral spread and is more of a qualitative than quantitative measure. Based on our review of the information and data presented, the site has an approximate 5 foot thick liquefiable layer that was encountered at a depth of approximately 10 feet, so that the site will have an approximate 10 foot thick confining layer, which will include a minimum 5 foot thick compacted fill blanket. Based on the procedure by Ishihara (1985), the site has a very low risk of ground surface manifestation. Since the site is relatively flat, the risk due to flow failure is considered unlikely. Any reduced bearing strength of the soils below the groundwater level is not expected to have a high risk on the structure, since the soil between the footings and the liquefied zone should provide an adequate bridge.

Settlement Due to Seismic Shaking

Granular soils, in particular, are susceptible to settlement during seismic shaking, whether the soils liquefy or not. Site processing, involving removal and recompaction of any shallow on-site soils that are loose and subject to seismically induced settlement, should effectively limit the potential for seismically induced settlement in these materials. The potential for earthquake-induced settlement was evaluated for the design-level seismic event using the procedures recommended by NCEER. This procedure is for relatively clean sands. Therefore, the blow counts were adjusted for the fine content [Stark/Olsen]. Additionally, blow counts were adjusted for borehole diameter, hammer energy ration, and sampling method.

Based on our seismically induced settlement calculations, the potential seismically induced settlement is estimated to be on the order of approximately 2.34 inches. Estimates equal to be about 1/2 the total settlement appears reasonable (Southern California Earthquake Center, 1999), due to the relatively uniform conditions of the deep alluvial sediments at the site. In this case, the average differential seismically induced settlement is estimated to be on the order of about 1.7 inches between adjacent supports spaced on the order of 30 feet. Your Structural Engineer should evaluate the consequences of such settlement to the proposed structures.

FOUNDATION AND SLAB MOVEMENTS

In addition to the settlement due to seismic shaking, foundation and slab movement will result from (1) the anticipated live and dead loads of the structure (2) the settlement of compacted fill and underlying soils due to the weight of the compacted fill, and (3) swell or hydroconsolidation if moisture changes occur within the supporting soils.

Settlement Due to Static Loads

Based on the information obtained from our geotechnical observations, static settlement is expected to be about 1-inch under the assumed loading conditions. Most of the settlement is expected soon after the application of the load. Additional foundation movements due to the weight of compacted fill or to soil swell (expansion) are expected to be negligible if the recommendations in this report are followed.

Settlement Due to Hydroconsolidation

Given our removal and recompaction recommendations and depth to groundwater, the surficial soils shall *not* be at risk of hydroconsolidation. Based on soil type, consolidation test results, and degrees of saturation, the soils below the removal and recompaction zone and above the groundwater level are considered to be at very low risk of hydroconsolidation.

Differential Movement

The amount of differential movement, including seismically induced (1.7 inches) and static (0.5 inches), between columns or adjacent footings due to the above causes and with mitigation measures included herein is expected to be about 2.2-inches in a lateral span of 30 feet. Such differential movement may possibly result in the development of cracks in the slab and in walls.

Slab Movement

As slabs are to be lightly loaded, the anticipated settlement is expected to be less than 0.25 inches under the proposed loading conditions.

CONCLUSIONS AND RECOMMENDATIONS

Based upon the findings of our site investigation, the site is considered to be suitable from a geotechnical engineering standpoint for the proposed multi-use site development. Based upon our understanding of the plans for development, the following recommendations are provided. Applicable elements of these recommendations shall be incorporated into the development plans.

Faults / Seismicity

Although no known active faults traverse through the subject site, like most of Southern California, the site lies within a seismically active area. Earthquake resistant structural design is recommended. Designing structures to be earthquake-proof is generally considered to be impractical, especially for private projects, due to cost limitations. Significant damage to structures may be unavoidable during large earthquakes. Structural design based on the 2019 CBC (California Building Code) structural analysis procedures calls for the seismic parameters given previously in the *Seismic Design Criteria* section. These minimum code values are intended to protect life and may not provide an acceptable level of protection against significant cosmetic damage and serious economic loss. Significantly higher than code parameters would be necessary to further reduce potential economic loss during a major seismic event. Structural Engineers, however, often regard higher than code values or procedures as impractical for use in structural design. The Structural Engineer and project owner must decide if the level of risk associated with code values is acceptable and, if not, to assign appropriate seismic values above code values for use in structural design.

The site is subject to liquefaction and seismically induced settlement, as mentioned previously. Buildings supported on continuous and mat foundations experienced less damage to the superstructure than buildings supported on individual footings without tie beams or with beams of low rigidity. Foundation or structural design measures, such as the use of continuous footings, mat foundations, structural slabs or deep foundations are mitigation measures. Other methods, implemented alone or in conjunction with structural design measures, used to mitigate the potential for liquefaction and related phenomena include vibro-replacement, vibro-compaction, compaction piles, chemical or compaction grouting, and dynamic compaction. These ground improvement mitigation methods, however, may not be cost-effective solutions. The owners may wish to accept the potential risk of liquefaction and associated damage and use shallow continuous footings and slab-on-grade with proper site preparation for foundation support, provided the Structural Engineer can show that the anticipated differential settlement, as discussed below, will *not* cause a structural collapse. The risk of damage to the proposed structure due to a large earthquake cannot be totally eliminated, and obtaining appropriate insurance as a mitigation measure is strongly recommended.

Hazardous Materials

This firm has *not* been retained to provide any type of environmental assessment of the subject property, *nor* to provide recommendations with respect to any contamination that might be present.

Landslides

Based on the results of our analysis of the proposed improvements with respect to the existing conditions, it is our professional opinion that the risk of landsliding at the subject site is *very low*.

Foundation Type

With proper site preparation, conventional shallow wall footings can be used for foundation support of walls, and spread footings can be used to support individual columns. Footings should be supported on compacted fill of relatively uniform thickness. Foundations for each structure should be totally founded in structural fill with a relatively uniform thickness and a minimum thickness of 3 feet below the footings. Garden walls can be supported on conventional wall footings.

The site is subject to liquefaction, seismically induced settlement, as mentioned previously. Buildings supported on continuous and mat foundations experienced less damage to the superstructure than buildings supported on individual footings without tie beams or with beams of low rigidity. Foundation or structural design measures, such as the use of continuous footings, mat foundations, structural slabs or deep foundations are mitigation measures. Shallow reinforced continuous footings and slabs-on-grade can be used from geotechnical engineering standpoint with proper site preparation for foundation support, provided the Structural Engineer can determine that the anticipated differential settlement, as discussed above, is within tolerable limits.

Removal Depths / Expansion Potential

Our exploration indicated that the strength and compressibility of the upper soils are variable, based on visual observations and on measured moisture and dry density variations. In our opinion, these near-surface soils are *not* suitable in their present condition for the support of structures or other improvements, without the potential for detrimental foundation movements occurring. Therefore, to mitigate these geotechnical hazards of the surficial soils, the upper soils will require removal, moisture conditioning, and recompaction *prior* to construction of the improvements. Recommendations for minimum removal depths are given below in the *Site Preparation* section, but surficial materials will need to be removed *prior* to placing compacted fill. Greater removal depths, however, may be required if the soils are wetter during construction than they were at the time of excavating the soil borings.

Site Grade Adjustments

A grading plan has *not* been provided as of the date of this report, but based on the proposed improvements, significant alteration of the existing grade is not anticipated.

Exploratory Excavations

The locations and dimensions of excavations completed during site exploration should be noted relative to the future grading/building plans. Although boring backfill was tamped during placement, these materials are essentially uncompacted. Removal and recompaction of these materials will be required for improvements over these excavations.

Excavation Characteristics

Difficult excavation in the locations of the proposed improvements should not be anticipated.

Shrinkage / Bulking

Shrinkage results when the soil/bedrock being placed as fill is compacted to a dry density greater than the in-place source materials, and bulking (negative shrinkage) occurs when the soil/bedrock is compacted to a dry density less than the in-place source materials. Based on experience, we estimate an average shrinkage factor of about 20% resulting from recompaction of on-site soils or fills. This estimate is based on an average relative compaction of 92% for recompacted materials and average densities of the undisturbed ring samples. The above shrinkage figures do not account for the effects of fill settlement losses due to clearing and grubbing and stripping operations, or uncertainty in the density of the in-place materials. If the actual average degree of compaction differs from that used to estimate shrinkage, the actual shrinkage may also differ. Variations in the estimated shrinkage/bulking factors shall be anticipated and provisions for such variations shall be included in the project specifications.

DRAINAGE

All surface runoff must be carefully controlled and must remain a crucial element of site maintenance. Proper drainage and irrigation are important to reduce the potential for damaging ground/foundation movements due to hydroconsolidation and soil expansion or shrinkage. Final grading shall provide a positive drainage away from footings in compliance with the local jurisdiction's grading requirements or a minimum gradient of 5%, whichever is greater, for a distance of at least 10 feet away from foundations for soil covered areas to reduce the risk of water ponding adjacent to foundations. For areas abutting foundations covered with concrete for a distance of at least 10 feet away from the foundations, a minimum gradient of 2% is acceptable. All pad drainage shall be collected and diverted away from proposed buildings and foundations in non-erosive devices. Gutters and roof drains are recommended and should be properly maintained, and discharge directly into glue-joined, watertight subsurface piping where feasible or released at a sufficient distance to prevent ponding of water adjacent to the foundation systems. Roof drainage should not be released in a concentrated manner. A drainage system consisting of area drains, catch basins, and connecting lines should be provided to capture landscape/hardscape sheet flow discharge water. All drainage piping should be watertight and discharge directly to concrete flatwork or paved areas serviced with area drains connecting to either the street or storm drain or discharge directly to the street or storm drain.

In the case of building walls retaining landscaping areas, a water proofing system should be used on the wall and joints, and a Miradrain drainage panel, or similar, should be placed over the water proofing. A perforated subdrain pipe of schedule 40 or better should be installed at the base of the wall below the floor slab and drained to the storm drain or curb. *Accordion* type pipe is *not* acceptable. Your project architect or Civil Engineer should provide detailed specifications for all waterproofing.

If a raised floor is used, the ground surface below the floor should be sloped away from footings and in a manner to collect and transfer any water due to a water line break, for example, to the street in a nonerosive device.

All underground plumbing fixtures should be absolutely leak-free. As part of the maintenance program, utility lines should be checked for leaks for early detection of water infiltrating the soils that could cause detrimental soil movements. Detected leaks should be promptly repaired. Proper drainage shall also be provided away from the building footings during construction. This is especially important when construction takes place during the rainy season.

Seepage of surface irrigation water or the spread of extensive root systems into the subgrade of footings, slabs, or pavements can cause differential movements and consequent distress in these structural elements. Trees and large shrubbery should *not* be planted so that roots grow under foundations and flatwork when they reach maturity. Landscaping and watering schedules should be planned with consideration for these potential problems.

Drainage systems should be well maintained, and care should be taken to *not over* or *under* irrigate the site. Landscape watering should be held to a minimum while maintaining a uniformly moist condition without allowing the soil to dry out. During extreme hot and dry periods, adequate watering may be necessary to keep soil from separating or pulling back from the foundations. Cracks in paved surfaces should be sealed to limit infiltration of surface waters.

Additional Recommendations

The following additional geotechnical recommendations should be incorporated into final design and construction practice. If the anticipated differential settlements are found by your Structural Engineer to be unacceptable some of the following recommendations may need to be modified. All such work and design should be in conformance with local governmental regulations or the recommendations contained herein, whichever are more restrictive. The following recommendations have *not* been reviewed or approved by the City at this time. These recommendations may change based on obtaining approval from the City. Design of the proposed project should be made following approval from the City.

Site Preparation

Based on available information, we understand that the site grade will not be significantly altered from the existing grade. The site is underlain by approximately 5 feet of surficial soils that are considered unsuitable for support of the planned structural loads. The building areas should be prepared by removing and recompacting the upper 5 feet of surficial soils. Building pads should be prepared so that the structure is totally founded in structural fill with a relatively uniform thickness of at least 3 feet, as measured from below the proposed foundation systems. General guidelines are presented below to provide a basis for quality control during site grading. We recommend that all structural fills be placed and compacted with engineering control under continuous observation and testing by this firm, and in accordance with the following requirements.

Removals

a. Remove all brush, vegetation and loose soil *prior* to fill placement. The general depth of stripping should be sufficiently deep to remove the root systems and organic topsoil. A careful search shall be made for subsurface trash, abandoned masonry, abandoned tanks and septic systems, and other debris (including uncertified fill) during grading. All such materials, which are *not* acceptable fill material, shall be removed *prior* to fill placement. The removal of trees and large shrubs should include complete removal of their root structures.

b. To reduce the risk of differential foundation movements, we recommend that all foundations be supported on structural fill, and the thickness of structural fill beneath the footings and slab area each be relatively uniform.

c. In all building areas, and areas to receive fill or to support slab-on-grade construction, the existing soil to a depth of 5 feet below the existing grade or a minimum of 5 feet below the bottom of the proposed slab, whichever is deeper, should be removed and recompacted as structural fill in the proposed construction areas. Furthermore, the surficial soils to a minimum depth of 3 feet below the bottom of the proposed footings should be removed and recompacted as structural fill. The maximum depth of recompaction below footings for garden walls or perimeter sound walls, however, can be limited to two feet. During construction where footings are in close proximity, over-excavating the entire structural area may be desirable and less costly.

d. In parking areas, driveways, and flatwork (patios, walkways) areas, a minimum of 24 inches below either existing grade or the structural section, whichever is deeper, shall be over-excavated and recompacted.

e. The removals can be limited to the proposed building, pavement, and fill areas but should extend a distance not less than 5 feet outside the slab-on-grade areas or fill limits, and 2 feet outside pavement areas, except in situations where a physical constraint, such as a property line or adjacent structure, would prevent such removals from being made. Removal limits for footings of buildings or accessory structures (e.g., garden walls) need only extend beyond the hardscape footprint a distance equal to the removal depth below the footing. A careful search shall be made for deeper loose soil spots during grading operations. If encountered, these loose spots should be properly removed to the firm underlying soil and properly backfilled and compacted as directed by a field representative of the Project Geotechnical Engineer. If the excavation to remove existing subsurface structures, pipelines, and loose fill soils extends below the minimum recommended depth of over-excavation, we recommend that all subsurface structures, utility lines, and uncontrolled fill extending below the over-excavation depth be removed to expose undisturbed, native soils across the entire building pad.

f. The exposed bottom of removal areas should be scarified, mixed, and moisture conditioned to a minimum depth of 8 inches. This thickness of scarification is included in the thickness of removal and recompaction mentioned above, unless the bottom is unstable and requires stabilization. The scarified soil shall be moisture condition to at least optimum moisture and compacted to a minimum of 90% of the laboratory maximum dry density as determined by ASTM D1557. Additional lifts should *not* be placed until the present lift has been tested and shown to meet the compaction requirements.

NOTE: The removal depths recommended herein are based upon our findings at the locations of our exploratory excavations. For the purpose of this report we can only assume continuity of subsurface conditions between the locations of our exploratory excavations in the area underlying the proposed building site. The thickness of unsuitable native soil may actually be more or less between the locations of our exploratory excavations than was observed at the test hole locations. Therefore, the total depth of removal of loose soils shall be determined in the field during grading operations, and it should be understood that actual depths of removal may be greater than discussed herein.

Bottom Stabilization

Due to the potential for high moisture content, and shallow groundwater, additional stabilization of the removal bottom may be required. If the bottom is unstable, the use of track-mounted equipment and/or excavators should be considered to reduce the potential for disturbing the soils in the excavations near the groundwater level. If the bottom is highly disturbed, deeper removals may be required. Acceptable stabilization methods include using (1) float rock worked into the soft soils and encapsulated with a filter fabric, (2) geofabric, such as Mirafi Fabric 600X, with a 24-inch wide overlap, or (3) a combination. Some compaction effort should be used when working thin lifts of float rock into the excavation bottom. A 12- to 24-inch thick zone may be required to adequately bridge an unstable bottom when using geofabric, and this zone is *not* to be included in the required thickness of fill beneath either slabs or footings unless it meets the compaction requirements. Another alternative is to stabilize the bottom by drying out the soils with the use of either lime or cement additives (about 5% by weight), moisture conditioning, mixing, and compacting to a minimum relative compaction of 90%. We recommend that unit rates for bottom stabilization be obtained during the construction bidding process, *prior* to construction.

Suitable Fill Material

a. The excavated site soils, cleaned of deleterious material, can be re-used for fill. Rock larger than 12 inches should *not* be buried or placed in compacted fill. Rock fragments less than 12 inches may be used provided the fragments are *not* placed in concentrated pockets or within 3 feet of final grade, and a sufficient percentage of finer grained material surrounds and infiltrates the rock voids. Furthermore, the placement of any rock must be under the continuous observation of the Geotechnical Engineer, and or his field representative.

b. Rock fragments greater than 3 inches may *not* be used within 6 inches of final grade.

c. Imported material should preferably have less than 15% by weight passing the number 200 sieve, a maximum plasticity index of 10, a liquid limit less than 25, and an expansion index of less than 50. Any imported soil from off-site sources shall be approved *prior* to placement.

Placement of Compacted Fill

a. All fill materials should be placed in controlled, horizontal layers *not* exceeding 6 to 8 inches thick and moisture conditioned to at least optimum moisture but no more than 5% above optimum. Fill materials should be compacted to a minimum 90% of the laboratory maximum dry density, as determined by ASTM D1557. If either the moisture content or relative compaction does *not* meet these criteria, the Contractor should rework the fill until it does meet the criteria. If the fill materials pump (flex) under the weight of construction equipment, difficulties in obtaining the required minimum compaction may be experienced. Therefore, if soil pumping occurs, it may be necessary to control the moisture content to a closer tolerance (e.g., 2 to 3% above optimum) or use construction equipment that is not as prone to cause pumping.

b. Each layer of fill under the building area within the upper 48 inches of the finished pad shall be of similar composition to provide a relatively uniform expansion index beneath the building. Selective grading shall be performed to either place more expansive soils in the deeper portion of the fill or to mix the more expansive soils with less expansive soils.

c. Subgrade for the support of pavement sections shall be moisture conditioned, as required, to obtain a moisture content of at least optimum but no more than 4% above optimum, and be recompacted to at least 95% of the maximum dry density to a depth of at least 12 inches.

Testing of Compacted Fill

At least one compaction test shall be performed for every 750 yd^3 of the fill material. In addition, at least one test shall be performed for every 2 feet of fill thickness.

Inclement Weather and Construction Delays

a. If construction delays or the weather result in the surface of the fill drying, the surface should be scarified and moisture conditioned before the next layer of fill is added. Each new layer of fill should be placed on a rough surface so planes of weakness are not created in the fill.

b. During periods of wet weather and before stopping work, all loose material shall be spread and compacted, surfaces shall be sloped to drain to areas where water can be removed, and erosion protection or drainage provisions shall be made in accordance with the plans provided by the Civil Engineer. After the rainy period, the Geotechnical Engineer and/or his field representative shall *review* the site for authorization to resume grading and to provide any specific recommendations that may be required. As a minimum, however, surface materials previously compacted before the wet weather shall be scarified, brought to the proper moisture content, and recompacted *prior* to placing additional fill.

c. During foundation construction, including any concrete flatwork, construction sequences should be scheduled to reduce the time interval between subgrade preparation and concrete placement to avoid drying and cracking of the subgrade or the surface should be covered or periodically wetted to prevent drying and cracking.

Responsibilities

a. Representative samples of material to be used as compacted fill should be analyzed in the laboratory by the Geotechnical Engineer to determine the physical properties of the materials. If any materials other than that previously tested are encountered during grading, the appropriate analysis of this material shall be conducted by the Geotechnical Engineer as soon as practicable. Any imported soil from off-site sources shall be approved *prior* to placement.

b. All grading work shall be observed and tested by the Project Geotechnical Engineer or their field representative to confirm proper site preparation, excavation, scarification, compaction of on-site soil, selection of satisfactory fill materials, and placement and compaction of fill. All removal areas and footing excavations shall be observed by the field representative of the Project Geotechnical Engineer before any fill or steel is placed.

c. The lateral limits and the depths of the removals should be shown by the Civil Engineer on the grading plans.

d. The grading contractor has the ultimate responsibility to achieve uniform compaction in accordance with the geotechnical report and grading specifications.

Utility Trench Backfill

The on-site soils are suitable for backfill of utility trenches from 1-foot above the top of the pipe to the surface, provided the material is free of organic matter and deleterious substances. The natural soils should provide a firm foundation for site utilities, but any soft or unstable material encountered at pipe invert should be removed and replaced with an adequate bedding material.

The site Civil Engineer in accordance with manufacturer's requirements should specify the type of bedding materials. If the on-site soils are *not* compatible with the pipe manufacturer's requirements, suitable nonexpansive, granular soils may need to be imported for bedding or shading of utilities. Jetting of bedding materials should *not* be permitted unless appropriate drainage is provided and the bedding has a sand equivalent greater than 50.

Trench backfill should be placed in 8-inch lifts, moisture conditioned to at least optimum but no more than 5% above the optimum moisture content, and compacted to at least 90% of the maximum density as determined by ASTM D1557, with the exception of the one foot below subgrade in areas to be paved, which should be compacted to 95% of the maximum dry density. If the contractor can demonstrate minimum compaction requirements can be achieved with thicker lifts, the acceptable lift thickness may be increased. Jetting of trench backfill is *not* acceptable to compact the backfill.

In areas where utility trenches pass through an existing pavement, the trench width at the surface shall be enlarged a minimum of 6 inches on each side to provide bearing on undisturbed material for the new base and paving section to match the existing section.

Major underground utilities shall *not* cross beneath buildings unless specifically approved by the Project Civil Engineer and respective utility company. If approved, trenches crossing building areas shall be backfilled with a select gravelly sand compacted to 95% relative compaction and at a moisture content at least optimum moisture but no more than 4% above optimum moisture.

Temporary Excavations

Temporary excavations of 5 feet or less in height in on-site soils may not require any special shoring. Vertical excavations more than 5 feet deep, if necessary, will, however, require conventional shoring per CAL/OSHA Regulations, or the excavation may be laid back with a 1(H):1(V) gradient. Excavations should *not* be allowed to become soaked with water or to dry out. Surcharge loads should *not* be permitted within a horizontal distance equal to the height of the excavation from the top of the excavation, unless the excavation is properly shored. Excavations that might extend below an imaginary plane inclined at 45 degrees below the edge of an existing foundation should be properly shored to maintain foundation support of the existing structure.

FOUNDATION SYSTEMS

Conventional spread footings founded into certified compacted fill can be used to support the proposed structure. The following foundation recommendations may be used in the design of conventional shallow footings.

Minimum Footing Dimensions

Structure	Minimum Exterior Footing Embedment Depth, Inches	Minimum Interior Footing Embedment Depth, Inches	Minimum Footing Width, Inches	Minimum Isolated or Spread Footing Width, Inches
Single-Story	12	12	12	18
Two-Story	18	18	18	24

The above embedment depths are below the lowest adjacent, final grade. Where located adjacent to utility trenches, footings shall extend below a one-to-one plane projected upward from the inside bottom of the trench. All isolated footing should be tied in a minimum of two directions with grade beams due to the potential for seismically induced settlement.

Allowable Bearing Pressure and Lateral Resistance

Allowable net vertical soil bearing pressure, including dead and live loads, are given below for footings founded on compacted fill at the minimum required embedment depths, provided the footing width equals or exceeds the recommended minimum. This allowable bearing value includes a safety factor of 3 or more and can be increased by ⅓ when considering short duration wind of seismic loads.

Support Material	Allowable Bearing Pressure, psf	Allowable Sliding Friction Coefficient	Allowable Passive Resistance, psf per foot of depth	Maximum Passive Resistance, psf
Compacted Fill	*2000*	*0.35*	*350*	*2000*

The above bearing value may be increased by 150 psf for each additional foot of footing width and 150 psf for each additional foot of embedment above the minimum to a maximum allowable bearing capacity of 2500 psf. The bearing capacity can be increased by ⅓ when considering short duration wind or seismic loads. Resistance to lateral loads can be assumed to be provided by friction along the base of the foundation and by passive earth pressures on the side of the footing. The allowable friction coefficient may be used with the vertical dead loads, and the allowable lateral passive pressure can be utilized for the sides of footings poured against the supporting material to resist lateral loads. These allowable values can be increased by a factor of 1.5 to convert from allowable to ultimate values. Where the soil on the resistance side of the passive wedge in not covered by a hard surface (e.g., concrete or pavement), however, the upper 1-foot of soil shall be neglected when computing resistance due to the potential for the material to become disturbed or degraded during the life of the structure.

Steel Reinforcement

All foundation should be reinforced with a minimum of four #4 steel bars. Two of these should be placed near the top of the foundation, and two should be placed near the bottom. Structural details of the footings, such as footing thickness, concrete strength, and amount of reinforcement, should be established by your Structural Engineer. The supporting soils have an expansion index category of very low (0-20). If the soil type encountered during grading differs from the specimen tested during this study, expansion index tests should be performed at the time of grading to confirm that more expansive soils are not present, and if they are present the designs may need to be revised. Due to the potential movements during seismic events and to the expansive nature of the soils, any spread footings should be structurally tied to wall footings with grade beams.

Required Observations

Prior to placing concrete in the footing excavations, an observation should be made by the representative of the Project Geotechnical Engineer to confirm that the footing excavations are free of loose and disturbed soils and are embedded in the recommended earth materials.

SLABS-ON-GRADE

Concrete slabs should be a minimum of 4 inches thick and reinforced with #4 rebar spaced at a maximum of 18 inches. If earthwork operations are conducted such that the construction sequence is not continuous or if construction operations disturb the surface soils, we recommend that the exposed subgrade to support concrete slabs be tested within a day of the concrete pour to verify adequate compaction and moisture conditions. The slab subgrade materials have an expansion category of very low (0-20). Due to the lightly loaded areas of exterior walkways and patio areas, soils with low expansion characteristics can lift such flatwork. This lifting will likely vary over the area covered by the flatwork, causing differential slab movements that could result in either a safety hazard or outwardly opening doors hanging up on elevated walkways that abut the structure. Therefore, we recommend that exterior walkways and patio areas abutting the structure where doors open outward with little vertical clearance be doweled into the structure at entrances and at joints to prevent differential movement of such flatwork due to soil expansion. Cracking of concrete flatwork can occur and is relatively common. Steel reinforcement and crack control joints are intended to reduce the risk of concrete slab cracking, as are the use of fiber reinforced concrete and proper concrete curing. If cracks develop in concrete slabs during construction (for example due to shrinkage), your Structural Engineer shall evaluate the integrity of the slab and determine if the design has been compromised. Also, concrete slabs are generally not perfectly level, but they should be within tolerances included in the project specifications.

Tile flooring can crack, reflecting cracks in the underlying concrete slab. Therefore, if tile flooring is used, the slab designer should consider additional steel reinforcement, above minimum requirements, in the design of concrete slab-on-grade where tile will be installed. Furthermore, the tile installer should consider installation methods, such as using a vinyl crack isolation membrane between the tile and concrete slab, to reduce the potential for tile cracking.