APPENDIX J GEOTECHNICAL INVESTIGATION

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DEPARTMENT OF PLANNING

Preliminary Geotechnical Investigation
Proposed Pacific City
Northeastern Corner of 1st Street
and Pacific Coast Highway
City of Huntington Beach, California.

November 19, 2001

PN 01039-00

Prepared For:

Mr. Ethan Thatcher
MAKAR PROPERTIES, LLC
4100 MacArthur Boulevard, Suite 200
Newport Beach, CA 92660

ZEISER
KLING
Consultants, Inc.

November 19, 2001 PN 01039-00

Mr. Ethan Thatcher
CAPITAL PACIFIC HOLDINGS, INC.
4100 MacArthur Boulevard, Suite 2000
Newport Beach, CA 92660

SUBJECT: Preliminary Geotechnical Investigation, Proposed Pacific City, Northeastern Corner of 1st Street and Pacific Coast Highway, City of Huntington Beach, California.

Dear Mr. Thatcher:

In accordance with your request and authorization, Zeiser Kling Consultants, Inc. (ZKCI) has completed this Preliminary Geotechnical Investigation using the 50-scale preliminary site plans prepared by McLarand Vazquez Emsick (the project Architect for the residential area), dated March 27, 2001, and plans prepared by Holmes and Narver (the project Architect for the hotel and commercial areas), dated September 21, 2001 for the proposed Pacific City project within the City of Huntington Beach, California. The accompanying report presents our findings, conclusions, and preliminary recommendations regarding the existing geotechnical conditions and their constraints on the design of the proposed development.

The findings and recommendations presented herein are considered valid as of the present date. However, changes in the conditions of a property can occur in the future, whether they are due to natural processes, the passage of time, or acts of man on this or adjacent properties. In addition, changes in applicable standards or codes may occur, whether from legislation or a broadening of knowledge.

The findings in this report are for planning purposes only, and will be refined in subsequent, site specific studies.

This report has been prepared specifically for the improvements associated with the Pacific City project. It has not been prepared for use by parties or projects other than specifically listed herein. It may not contain sufficient information for other parties or other purposes. This report is also subject to review and acceptance by the City of Huntington Beach.

This report is also subject to the limitations presented in Section 11.0 of our report and the ASFE (the Association of Engineering Firms Practicing in the Geosciences) insert included in Appendix J.

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The opportunity to be of continued service to Capitol Pacific Holdings, Inc. is appreciated. Please contact the undersigned with questions or comments.

Respectfully submitted,

ZEISER KLING CONSULTANTS, INC.

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1.0 INTRODUCTION

1.1 Executive Summary

- The general distribution of the geologic materials is roughly as described in the previous consultant reports reviewed.

- The findings with respect to engineering support capacity of the on-site materials have been evaluated with significant detail based on both surface and subsurface explorations. The following are believed to apply:

  - The proposed development is feasible from a geotechnical point of view.
  - The site is not located within a Fault Hazard Zone as defined by the State of California and the City of Huntington Beach. The southwestern third of the site is located within a Seismic Hazard Zone for Liquefaction as defined by the State.
  - The site is within an area of Tsunami Run-up as defined by the City General Plan. Up to 8-feet of run-up may be expected during a 500-year seismic event.
  - The soils within the Talbert (also known as the Santa Ana) Gap area in the southwestern portion of the site are prone to consolidation and have a moderate potential for liquefaction induced settlement.
  - Due to the weak soil conditions in the southeastern portion of the site, deep and stiffened foundations are recommended for buildings over and under two-stores, respectively.
  - The terrace deposits exhibit a "medium" to "high" potential for expansion; a "negligible" to "moderate potential for corrosion towards concrete elements; and a "severe" potential for corrosion towards ferrous metals.

- The current grading scheme appears at this time to be feasible, provided the geologic and groundwater conditions are taken into account. Recommendations for such are presented herein. However, the following special provisions will apply:

  - Existing remnants of structures, such as the slabs and foundations for the former motel, within the grading limits should be demolished and removed from the site.
  - Abandoned oil wells near or within proposed buildings will require special venting in accordance with Huntington Beach Specifications 429 and 431 (See Appendix F).
  - The proposed depth of the parking lot floors are at or within a few feet of the groundwater table. This condition will most likely require some dewatering or other engineering control of excavations during construction.

- To mitigate the potential for transient groundwater conditions, the proposed buildings should be underlain by a underdrainage system to prevent hydrostatic build-up.
- To facilitate compaction of the terrace materials, and to limit the expansion potentials, the fills are recommended to be placed at a minimum of 120-percent over the optimum moisture content.

The conclusions and recommendations presented in this report are preliminary, and are subject to revision in future site- and building-specific studies. This report is intended for planning purposes only and is to be used for the Environmental Impact Report (EIR) currently being prepared for the site.

1.2 General Intent and Scope

This report presents the results of our study of the geotechnical and engineering geologic aspects of the site with respect to the proposed development. The findings of this report are intended to be incorporated into the "Earth Resources" Environmental Impact Report (EIR) for the subject proposed development in accordance with California Environmental Quality Act (CEQA) and National Environmental Protection Act (NEPA) requirements. This report also presents general and preliminary geotechnical recommendations with respect to grading and foundation systems for the proposed development.

The general scope of work performed included:

- Review of previous engineering and other scientific studies performed on the site;
- Review of the site history, as well as several series of aerial and aerial stereo photographs of the site and vicinity from 1953 to 1994;
- Performance of reconnaissance mapping of the site;
- Performance of six 5-inch diameter mud-rotary borings which penetrated and sampled the engineering materials underlying and immediately supporting the site;
- Conversion of two of the borings (ZB-2 and ZB-5) into groundwater monitoring wells (piezometers).
- Laboratory testing in support of the field exploration to evaluate pertinent engineering characteristics of the underlying soils;

- Compilation of the findings and data;
- Development and evaluation of models of the site with respect to engineering geologic structure;
- Evaluation of the proposed development relative to existing conditions; and
- Preparation of this report, presenting our findings, conclusions, and recommendations. The engineering geology of the site and locations of field explorations are presented on Plate I.

The base topography for our Geologic Map, Plate I, was prepared by Hunsaker and Associates (the project Civil Engineer). We are including a copy of the plan prepared by McLarand-Vasquez-Emsick (MVE) as Plate II, Site Plan, showing the locations of the borings with respect to the proposed building locations. The list of reference data is presented in Appendix A. Our study of the site was supplemented by surface and subsurface data garnered by other competent geotechnical firms. It is recognized that the findings and recommendations presented herein are subject to jurisdictional review and approval, as well will be supplemented by future foundation specific studies.

The purpose of this phase of study is to provide information regarding geotechnical feasibility, and information to be used in a Environmental Impact (E.I.R.) level report.

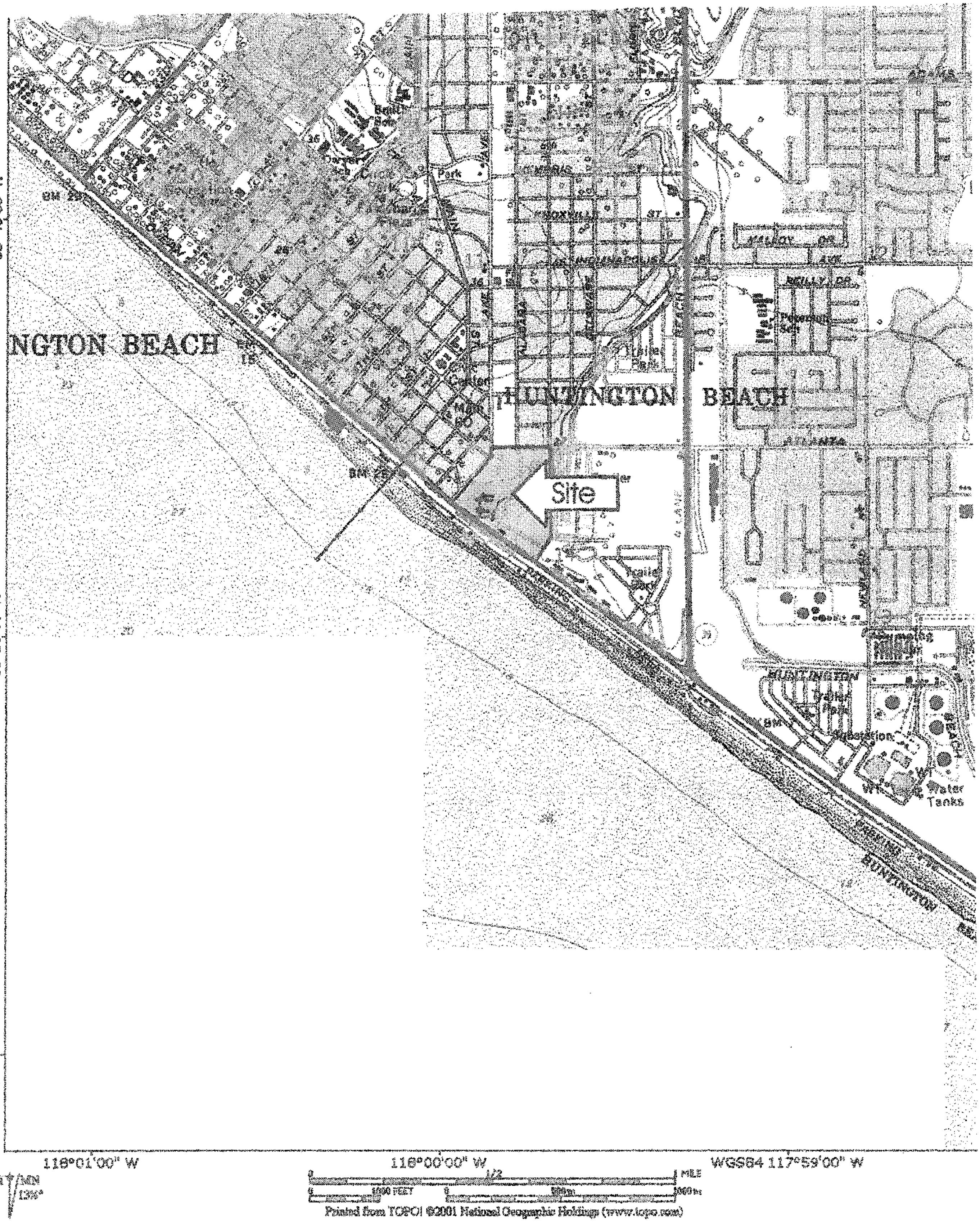
1.3 Site Location

The site is located in the southwestern portion of Huntington Beach. Specifically, the site is located adjacent and east of the Pacific Coast Highway (PCH), and is bounded by Huntington Street to the east, Atlanta Avenue to the north and 1st Street to the west.

The site location is presented on Figure 1, and the site layout is shown on Plates I and II, with the proposed development being shown on Plate III.

1.4 Proposed Development

Based on our site reconnaissance and our meetings with you, we understand that the site is currently proposed for use as an integrated development including entertainment, hotel, residential, and restaurant/retail centered on the "Pacific City" theme.



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ZEISER KLING Consultants, Inc.	**Site Index Map** Proposed Pacific City NEC Pacific Coast Hwy and 1st Street HUNTINGTON BEACH, CALIFORNIA	FIGURE: 1 PN: 01039-00 DATE: NOVEMBER 2001

Pacific City is proposed to include:

- Development of one mid-rise hotel complex (8-levels with 2-levels of subterranean parking);

- Development of entertainment, retail and restaurant facilities facing Pacific Coast Highway within the western third of the site (with 2-levels of subterranean parking);

- Two 6- to 12-story mid-rise condominiums located in the central portion of the site (with 2-levels of subterranean parking);

- Several residential buildings (3- to 4-levels with 1-level of subterranean parking) along the northern, eastern and southern portions of the eastern two-thirds of the site; and,

- Associated roadways and infrastructure.

As a part of our study, we have been provided with a 50-scale site plan for the residential portion of the site, dated March 27, 2001 and prepared by MVE; a conceptual 50-scale site plan for the commercial and hotel portion of the site, dated September 21, 2001 and prepared by Holmes and Narver; and, the Pacific City Concept Brochure, prepared by Makar Properties.

2.0 SITE CONDITIONS AND HISTORY

2.1 Site Description

The proposed Pacific City development is located approximately three blocks south of the Huntington Beach Pier, on the northeastern corner of the Pacific Coast Highway (PCH), and 1st Street. The Waterfront Hilton is located to the east of the subject site, across from Huntington Street. Downtown Huntington Beach and the Huntington Beach pier are located to the west of the subject site.

The site is irregular in shape, approximately 34-acres in size. There is evidence of previous oil drilling which have resulted in alterations to the previous landform. The most notable changes are berms, oil wells, and sumps constructed as part of the oil drilling operations.

We observed some fire hydrants and standing light poles within and around the former Huntington Shores Motel parking lot. It is unknown if they are tied into live utilities. It is also our understanding that an active 18-inch water main is running across the site, parallel and approximately 45 feet from the centerline of PCH. The location of this line should be confirmed prior to grading.

The city is utilizing the parking lot for the former Grinder Restaurant and similar pre-existing developments, to store and stage city vehicles. Also, remnants of slabs-on-grade, foundations and asphalt pavement from the former Huntington Shores Motel are present. With the exception of the remnants of the former motel and the city parking lot in the southwestern corner of the lot, the site is in a generally cleared condition.

The northern portion of the site has recently been used as a borrow pit, which has resulted in a topographic depression approximately up to 20 feet below previously existing grade.

There were no signs of heavy vegetation, trees, structures, standing or running water, wetlands, active oil wells or other similar features observed at the site during our field reconnaissance or exploration.

2.2 Site History

In reviewing the previous reports, the following information was determined. This information is supplemented with observations made by our firm in a review of aerial photos from 1952 to 1994.

The project site is located within the "Huntington Beach Oil Field," that was operated by Chevron. Although the site was used as an operating oil field, it has been shut down for many years.

The southwestern portion of the site, along PCH also had some previous development. In the 1952 aerial photograph, there was a parking lot in the northwestern portion of the site for the beach. In addition, portions of the site have been reported to have been used by the Pacific Electric Railway alignment.

In the 1970 aerial photograph, the Grinder Restaurant, the Huntington Shores Motel and a trailer park were located along PCH. The northern half of the site was vacant, with scattered oil wells.

In 1999, approximately 200,000 cubic yards of material was exported from the northwestern two-thirds of the site. This material was removed from August to October 1999, and utilized as export for the on-going Ocean Grand Hilton Resort project. The bottom of this pit was backfilled in August 2000 with approximately 2 feet of soil.

2.3 Offsite Developments and Conditions

A review of geotechnical and related reports of the surrounding developments was conducted where reports were available. The following information was obtained from these reports. A detailed list of these reports is included within Appendix A.

The Hilton Waterfront Beach Resort hotel is located to the southeast of the site. Currently construction is underway for the Ocean Grand Hilton Resort, which will extend the Hotel to the southeast along Pacific Coast Highway. Huntington Beach State Park is located south of the site across from Pacific Coast Highway.

Downtown Huntington Beach, consisting of several retail and commercial buildings are located west-northwest of the site. The Huntington Beach Pier and Main Street is approximately 3 blocks northwest of the site. Residential homes and apartments are located north of the site and a trailer park is located east of the site.

3.0 PREVIOUS STUDIES

3.1 Engineering Reports

The site was previously studied by Stoney-Miller Consultants, Inc. (SMC), Levine-Fricke (LF), Pacific Soils Engineering, Inc. (PSE) and AGRA Earth and Environmental (AGRA). The previous studies were performed from January 1989 through April 1998.

- Stoney-Miller Consultants, Inc. performed a feasibility study (Reference 39) of the site in 1989. Their study was titled "Geotechnical Feasibility Investigation, Approximately Seven Acre Huntington Street Property, Huntington Beach, California," dated January 30, 1989. This study included general reconnaissance mapping and subsurface exploration of the southeastern portion of the site to provide recommendations for the placement of approximately 12-feet of fill in this area to bring this portion of the site to bring the site to a relatively level grade.

  The subsurface work included the advancement of eleven 8-inch diameter hollow-stem auger borings. The locations of all the borings performed in the SMC report are shown on Plate I, Geologic Map, and their logs are included in Appendix C.

  SMC concluded that the natural soils (alluvial) were generally suitable for placement of fill. However, due to the compressible nature of the alluvial soils, they recommended an overexcavation of 5- to 6-feet across the site. They encountered groundwater within the alluvial units at depths ranging from 3- to 7-feet below the ground surface.

  Based on our observations of the subject site, the proposed grading discussed in the SMC study was apparently not conducted.

- Levine Fricke (LF) performed a preliminary study (Reference 32) of the site in 1996. Their study was titled "Report of Preliminary Geotechnical Study, Morgan Stanley Atlanta Property, Huntington Beach, California," dated July 22, 1996. This study included a review of the 1987 SMC report and subsurface exploration of the site to provide recommendations for grading and foundations for proposed residential homes, three to four story apartments, and related below-grade parking garages.

  The subsurface work included the advancement of eight Cone Penetrometer Test (CPT) probes across the site. The locations of the CPT soundings performed in the LF report are shown on Plate I, Geologic Map. CPT logs are included in Appendix C.

LF concluded that the Terrace deposits were generally suitable for placement as fill and for foundations. However, due to the compressible nature of the alluvial soils in the southeastern third of the site, they recommended stiffened foundations for two-story residential wood structures, and deep foundations for buildings over two-stories.

- Pacific Soils Engineering, Inc. (PSE) performed a feasibility study (Reference 34) of the site in 1997. Their study was titled "Feasibility-Level Geotechnical Study, Atlanta Development, Pacific Coast Highway and First Street, City of Huntington Beach, California," dated October 27, 1997. This study included surface mapping and subsurface exploration of the site to provide recommendations for grading and foundations for proposed residential homes, three-story apartments and retail/mixed use area with related below-grade parking garages, and three 7- to 8-story buildings.

  The subsurface work included the advancement of ten 8-inch diameter hollow-stem auger borings, 12 test pits and five CPT soundings across the site. The locations of all the CPT soundings performed in the PSE report are shown on Plate I, Geologic Map. CPT logs are included in Appendix C.

  PSE concluded that the upper 20 to 30 feet of alluvial soils beneath the groundwater table in the southeastern portion of the site are compressible. Based on the 25-feet of fill proposed for this development concept, PSE estimated that up to 24-inches of settlement may occur. Their seismic settlement analysis estimated that liquefaction related settlement should range from 1/2 to 1-inch in addition to consolidation related settlement. Additionally, terrace deposits exhibited a "medium" to "high" potential for expansion, a "negligible" to "moderate" corrosion potential towards concrete and "severely" corrosive towards ferrous metals. They encountered groundwater within the alluvial and terrace units at depths ranging from 5- to 10-feet (elevations of approximately 3 to –5 feet MSL) below the ground surface.

- In 1998, PSE produced another geotechnical report to address the remedial grading plan review (Reference 33) for the development discussed in their 1987 study. They did not perform any additional subsurface investigation for this report. PSE concluded that the settlement over the alluvium in the southeastern portion of the site would be approximately ½-inch per foot of fill placed. They estimated that this would occur within 4- to 6-months after the completion of rough grading, and that the settlement should be monitored.

- The most recent previous study (Reference 5) was performed by AGRA was titled "Summary of Findings, Environmental and Geotechnical Site Investigation, Ocean Front Plaza, Huntington Beach, California," dated April 1, 1998. It appears that AGRA advanced a total of three 5-inch diameter mud-rotary wash borings and three push-sample ("geoprobe") borings within the site to confirm findings presented in the PSE report.

AGRA concluded that based on a isobath map they reviewed, that groundwater may be encountered within 3-feet of the ground surface in the southeastern portion of the site and at a depth of 30 feet in the northwestern portion of the site.

SMC, LF, PSE and AGRA generally found the site to be generally as depicted in this report. The results of these studies were incorporated into this report and were used to supplement our independent information. The findings of these consultants reports served as a starting point for our evaluation.

3.2 Compaction Reports

Sometime between the PSE 1997 report and August 2000, the northwestern portion of the site was used as a borrow site for use at the neighboring Ocean Grand Hilton project. Approximately 30 to 35 feet of material was removed, forming a large pit. During the month of August, the bottom of this pit was backfilled with approximately 2 to 3 feet of compacted fill. The grading was performed under the observation of AGRA, and the compaction test results are presented in their August 21, 2000 report (Reference 1) titled "Observation and Testing Services During Backfill of Borrow Area, Atlanta-Huntington Beach Development, Huntington Beach, California." This report does not describe the method of placement, depth of placement or the source of the fill material, and provides only laboratory and density test results.

3.3 Environmental Reports

Due to its past use as a operating oil field, numerous environmental characterization and remediation reports have been performed by Harding ESE (formerly Harding Lawson and Associates) and AGRA. These reports were written between December 1996 and January 2001.

Currently, the environmental consultant for the previous property owner (Chevron) is planning to implement a remediation plan that will involve removal of oil-contaminated soils, remediation of these soil on site, and replacement of the soil removal areas with compacted fill materials.

Reports and remediation plans have been reviewed for any information regarding the history of the site and adjacent areas, and subsurface information that may be useful from a geotechnical standpoint. It is our understanding that the environmental issues and remediation of the site is being performed by representatives of the previous owner.

4.0 FIELD AND LABORATORY TESTING

4.1 Surficial Field Work Performed in this Study

Field work performed in this study included geologic mapping of surficial exposures of geologic conditions at the site. The attached Geologic Map, Plate I, presents the results of the field mapping and shows the location of the current and past subsurface exploration within the subject site.

4.2 Subsurface Exploration Performed in this Study

Our subsurface exploration included the advancement of six 5-inch diameter mud-rotary wash borings. These borings were advanced to depths ranging from approximately 76.0 to 101.5 feet below existing grade. The borings were intended to allow us to determine the depths of the artificial fill and terrace materials. Mud-rotary borings are best suited for sampling in sandy soils with shallow groundwater, since the drilling mud applies a constant head within the boring and counteracts heaving sand conditions.

The borings were also used for obtaining representative samples of the subsurface soil and formational materials. These relatively undisturbed and bulk samples were recovered, sealed and transported to our laboratory for classification and testing.

The locations of the borings are shown on the attached Plate I, Geologic Map. Logs of the borings are presented in Appendix B. Exploration logs from previous geotechnical studies of the site are presented in Appendix C.

4.3 Laboratory Testing

Representative samples of the subsurface conditions were tested in our laboratory to determine soil classifications and pertinent engineering properties. The test results with respect to moisture/density are presented on the boring logs in Appendix B. The detailed laboratory test results and discussion are presented in Appendix D.

The test results presented in the previous reports, as discussed in Section 3.1, are presented in Appendix E.

5.0 GEOLOGIC SETTING

5.1 Regional Geology

Introduction

The Pacific City development, as is Orange County in general, is a portion of the Peninsular Ranges Geomorphic Province. The Peninsular Ranges are distinguished by series of strike-slip fault controlled mountains and other structure belts that trend north-northwest. The northern/northwestern portion of the Peninsular Ranges, of which the Pacific City development and the general Huntington Beach area is situated, is a relatively broad, somewhat irregular coastal plain.

Geomorphically, the site is situated in the northerly/northwesterly fringe of the Talbert Gap (also known as the Santa Ana Gap) and the southerly limit of the Huntington Beach terrace mesa. The Talbert Gap, along with Bolsa and Los Alamitos Gaps located to the northwest, are the result of the combination of downcutting and subsequent flooding/depositions. These mesas represent the remaining portions of a now strongly dissected coastal terrace. The terrace materials forming the mesas are comprised of relatively well consolidated to slightly indurated marine and terrigenous sediments of a predominately fine granular nature. The material of the gaps are notably less consolidated, in general being only normally consolidated, contain significant silty fines and zones of peat, and have prevalent groundwater and saturated zones of a relatively shallow nature. The transition between the mesa and gap terrain is typically distinct topographically.

The dominant structural geologic feature controlling the area of the site is the Newport-Inglewood Fault. Activity on this fault, combined with regional tectonic effects (such as uplift) have combined with climatic forces and changes in sea level since Pliocene to Pleistocene (the past 2 to 3 million years) time to form the underlying basement materials and structure that underlay and support the site. The forces that have created the geomorphology of the site and vicinity are still active today.

No faults are known to cross or underlie the site. It is, along with the general vicinity, underlain at depth by a "structural zone" forming the Huntington Beach Oil Field. No ground ruptures are known to have occurred onsite in response to groundshaking induced by offsite faulting, including the 1933 Long Beach earthquakes.

The site is located very near the south and north branches of the Newport-Inglewood Fault Zone. These branches strike very nearly northwest and are of very high angle to near vertical. The seismic nature and influence of this fault is discussed in Section 5.3 of this report.

Pertinent Geologic History

The majority of the terrace deposits were laid in a shallow marine to near-shore terrestrial environment on an unconformable bedrock platform in the Pleistocene time frame (i.e. about 2-million to about 10-thousand years ago). The source of these sediments was erosion of the rocky highlands of San Bernardino, Santa Ana and other mountain belts. The Santa Ana River and other fluvial systems deposited most of these sediments that were reworked by shallow marine and coastal processes. These deposits developed into a thick regional unit.

These deposits were then later uplifted by tectonic forces associated with regional faulting that includes the Newport-Inglewood, San Andreas, and offshore deformation zones. This uplift exposed the terrace materials to erosion, removing much of their cover, which is the general source of their overconsolidation. The terrace materials were then dissected by action of coastal plain rivers/streams starting in late Pleistocene time, forming the topographic prism of the "gaps". These streams included the proto-Santa Ana River, which carved out the Talbert Gap. These gaps were later infilled by sediments of a fluvial and shallow to lagoonal/estuarine marine nature. The dissection and infilling of the gaps was predominately a function of interaction of uplift of the area, and fluctuations in sea level. The fluctuations in sea level were a function of the "ice-ages," where periods of glaciation dropped sea level. When sea level dropped, downcutting of the gaps was promoted. When sea level rose, deposition was promoted, the rivers turned stagnant and eventually drowned, becoming slough-like and eventually lagoonal or estuarine. These processes have continued into recent time, until interrupted by actions of man and development, and channelization of the drainages and rivers.

Recent man-made developments and activities have modified the site. Oil field development and infrastructure was installed at several locations across the site since as early as the 1920's. Nearly all of these oil wells have been abandoned and the site substantially remediated with respect to those features by others prior to our involvement with the site, however, some features of the oil operations and associated infrastructure are anticipated to still be present.

The beachfront portion of the site fronting Pacific Coast Highway has had extensive small-scale development in the form of a motel, restaurants, trailer park and other features described elsewhere in this report. These developments included grading which have placed some amounts of artificial fills in these areas. Although these developments have been demolished, the fills and foundational remnants of the buildings remain.

The site was also modified in the northerly portion by borrow-grading in the recent past (circa mid-2000). This grading has resulted in the formation of a "pit" area. The specifics of this grading are also described elsewhere in this report.

Groundwater

Groundwater is a prevalent condition in the general area of the site. Although several bodies of groundwater are known to exist, particularly in the gap terrain, only the shallow groundwater members which will have an affect on the engineering and construction aspects of the project are considered in this study.

Free groundwater was encountered in all the ZKCI borings and other consultant exploration points that extended to about sea level elevation. The generalized groundwater elevations are within a few feet of sea level. The average elevation onsite is about 2 to 4 feet above sea level. Localized perched zones, and areas subject to concentrated climatic effects or surface water channeling may cause localized higher areas of seepage or groundwater. The surficial/near surface groundwater is essentially an unconfined aquifer system. It may have some response to localized climatic effects (i.e. intense prolonged rainfall, strong prolonged drought, or similar) that may temporarily change the water table on a limited basis.

The groundwater considered in this study, although of an overall generally unconfined nature, is believed controlled to at least some extent by stratigraphic considerations and preferential permeability. This is believed to be particularly applicable to the alluvial materials associated with the gap terrain in the south to southwest portion of the site.

These stratigraphic conditions may be expected to cause this shallow, overall essentially unconfined groundwater to behave in a more semi-confined to confined manner on a localized basis. This is expected to be especially applicable under the conditions that the groundwater is anticipated to be encountered in the course of construction. Our understanding of the stratigraphy suggests the groundwater behaves as a series of anastomosing or interlensed stackings of semi confined and confined zones of higher transmissivity, separated by zones of lower permeability. The net effect is a condition where lateral permeability and transmissivity following the stratigraphic "grain" is markedly higher than moving vertically across this "grain."

Our understanding of the current development scheme is that the basement / lowest finished floor elevations of the proposed buildings will be very close to the existing groundwater elevations. Because of the significance of groundwater on constructability and long-term performance, given such considerations, a detailed site-specific study for each structure that addresses the effects of groundwater should be performed. Such specific study is beyond the scope of this report.

We understand that the environmental issues with regard to contamination of groundwater have been addressed and effectively remediated by other consultants. Although environmental evaluation was not a part of our study, no obvious evidence of contamination was encountered in our exploration of the site.

Review of the AGRA data and associated testing indicates that much of the shallow groundwater that may influence the proposed development is of a fresh water nature and supports fish life. Some areas of brackish to very salty water also exist, and are believed related to concentrations of lagoonal and tidal-flat infiltrations from ancient times, and other saltwater sources.

No surface water, wetlands, or other non-storm drainage related hydrologic features were observed at the site. No springs or seeps were encountered on the exposed surface. Surface drainage existing onsite in response to precipitation is generally of a sheet flow nature, except where controlled otherwise by topography or erosion control devices.

5.2 Geologic Units

In the regional vicinity of the subject site, the underlying materials are Artificial Fills (Afu), Quaternary/ recent alluvium deposits (Qal) and Pleistocene Terrace deposits (Qtm).

Generalized descriptions of these units are:

5.2.1 Artificial fill (Af)

In general, artificial Fill materials are materials placed by man, whether engineered or dumped on a site. The majority of the fills placed at this site are essentially undocumented, with the exception of the fill placed under the observation of AGRA in August 2000 in the borrow pit area. This fill is present on the majority of the middle of the northern portion of the site and along the northern edge of PCH. These fills are discussed in detail below:

<u>Artificial Fills (Afu):</u>
Artificial fill defines the undocumented and/or uncontrolled fills that were placed in several local areas of the site. These fills are generally silty sands, sandy silts, silty clays and silty clays that are found along the northern side of PCH and are

associated with the former hotel, restaurant and trailer park. There are also other isolated fill materials across the site related to the oil field operations.

It is our understanding that there is an archaeological site consisting of debris from the former Pacific Electric Rail Line operations buried in the fill in the southeastern portion of the site.

Engineered Fills (Afc):
Engineered fill defines the documented fills that were placed in select local areas of the site. These fills are generally silty sands to clayey sands that are found within the bottom or near the "borrow pit" in the northwestern two-thirds of the site. The compaction of this fill is documented in the August 21, 2000 AGRA report.

5.2.2 Sedimentary Units (Qal and Qtm)

Sedimentary materials are soils that are generally deposited by water. The alluvial units within the site are found within generally low-lying locations. We subdivided general alluvium into two categories for clarification and analyzing distinct properties.

Alluvium (Qal_y):

The younger alluvial, young coastal, lagoonal and estuarine deposits associated with the gap terrain are all broadly similar in engineering and foundation character and occurrence, and thusly are included in this material designation. As discussed elsewhere in this report, these materials are generally found within the southeastern third of the site. These materials within the site is also characterized by zones of brown to gray sandy clay to sandy silt, and clayey sands to clayey silt with lenses and zones of silty and poorly graded sands. The estuarine/lagoonal-derived materials may contain fossil zones including small shell remnants.

The structure of these gap materials is generally lensatic to crudely interstratified. Since both fluvial processes (i.e. flowing water-related) and coastal/shallow marine processes have operated over time to place these deposits, interfingering, local anomalous/unconformable horizons, cross-cutting and pinchouts are likely typical.

Alluvial materials of a limited nature may also be found typically as recent deposits within localized topographic lows within the subject site. These deposits are very limited in extend and nature and are not shown on the geologic map.

Terrace Deposits (Qtm):

Terrace deposits constitute the oldest surficial deposit on-site. These units represent alluvial and shallow marine/coastal deposits that have been consolidated and overconsolidated in the past, experienced regional uplift, and eroded down to its present state from previously much thicker regional deposits. This process provides a material that is typically well suited for foundation support.

The terrace deposits consist of reddish brown to brown, locally yellowish to grayish, generally over-consolidated, interlayered lenses of silty to clayey sands, clayey silt and silty clay with some interbeds of gravel and cobbles, that are generally slightly moist to moist and dense to very dense. It should be noted that some zones of clayey soils and zones and lenses of less indurated, softer sediments may occur within the terrace. These softer sediments, although of an overconsolidated nature by virtue of geologic history, represent localized conditions that may react adversely to relatively heavy foundation loading. At this time, these zones are considered untrustworthy with respect to heavy bearing capacity, and deep foundation members for the proposed buildings should be extended through them. The specific aspects of these softer zones and their interactions with the proposed foundations will be addressed in detail in forthcoming foundation specific studies.

5.2.3 Structure

Since the subject site is located on the fringe of both the terrace and the gap terrains, it has the aspects of both. In general, the northerly and easterly portions of the site are terrace terrain, and are underlain by consolidated terrace deposits. The southerly to southwest portion is underlain by the fringe of the gap terrain, and is underlain by a wedge of softer, more poorly consolidated sediments that include alluvial and lagoonal deposits. The thickness of this wedge of gap deposits increases to the south and south east, eventually becoming very deep offsite in the mid-gap areas. Underlying the wedge of gap soils onsite is a basement of terrace deposits of a well-consolidated nature.

The limits of the geologic units are depicted in the attached Geologic Map, Plate 1. The structure is depicted in the cross sections, Plates III and IV. The character of the subsurface materials is described in the boring logs presented in the appendix.

5.3 GEOLOGIC HAZARDS

5.3.1 Faulting and Seismicity

The subject site, as already discussed, is within the Huntington Mesa at the northwestern edge of the Talbert Gap (also known as the Santa Ana Gap). The Santa Ana Basin and the Huntington Beach area were formed as a result of regional uplift along the Newport-Inglewood Structural Zone, and downcutting as a result of erosion from the Santa Ana River floodplain and littoral processes along the coastline. As a result, a system of mesas and gaps have developed along the coast in this area.

Hazards associated with seismic activity include primary hazards, such as ground shaking and surface rupture, and secondary hazards including liquefaction, seismic settlement, seismically induced landsliding, tsunami, and seiches.

The California Division of Mines and Geology defines an "*active*" fault as a fault that has shown evidence of activity within the last 11,000 years. A fault that has experienced activity within the last 2 to 3 million years, but has not shown direct evidence of activity within the last 11,000 years is defined as "*potentially active*". An "*inactive*" fault is defined as a fault that has not experienced activity in the last 3 million years.

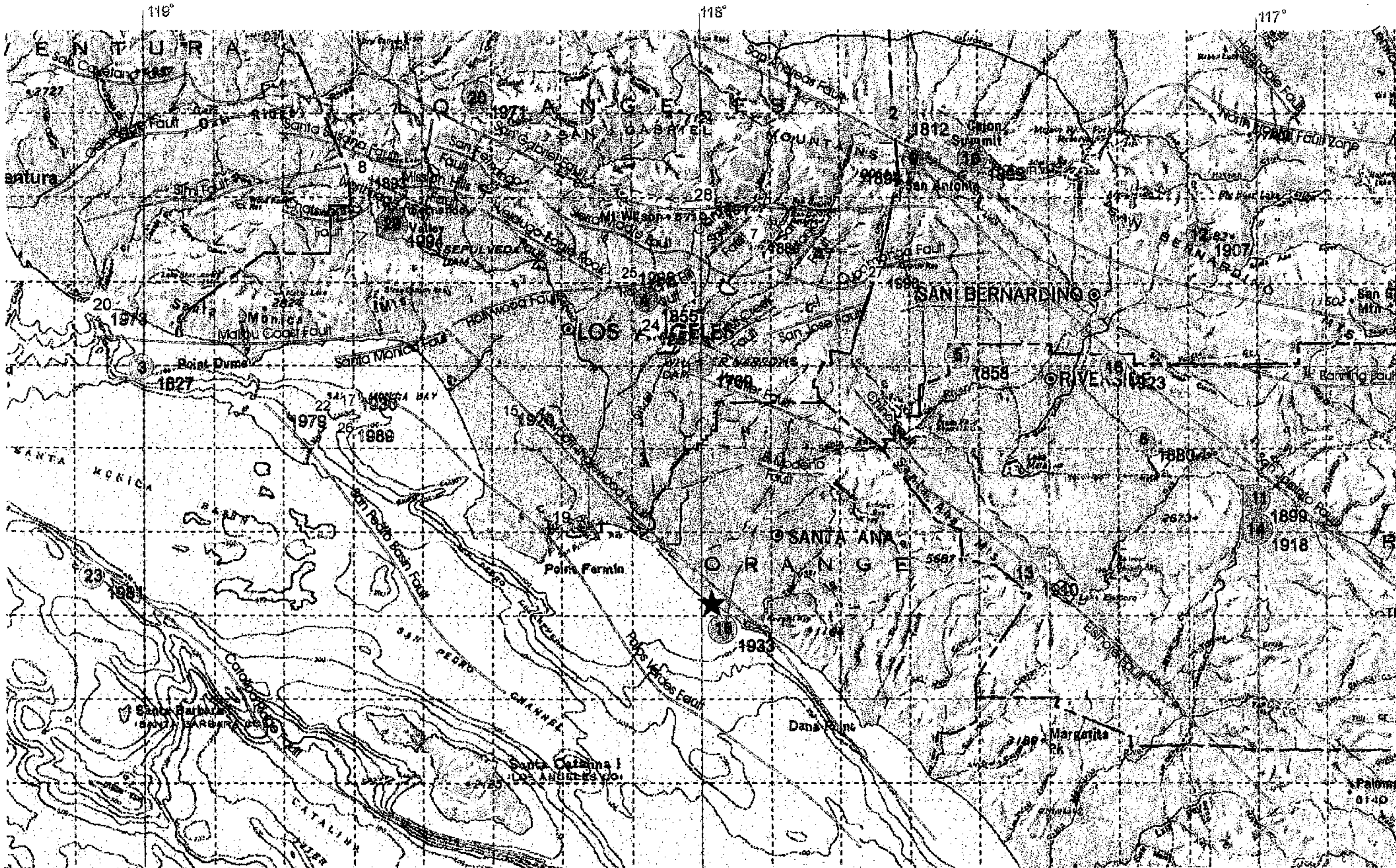
The Alquist Priolo Act of 1972 (now the Alquist Priolo Earthquake Fault Zoning Act, Public Resources Code 2621-2624, Division 2, Chapter 7.5) regulates development near active faults so as to mitigate the effect of surface fault rupture. Under the act, the State Geologist is required to delineate "Special Studies Zones" along known active faults in California. The Act also requires that, prior to approval of a project, a geologic study be conducted to define and delineate any hazards from surface rupture. A geologist registered by the State of California, must prepare this geologic report. A minimum 50-foot setback from any known trace of an active fault is required.

Active faulting is not believed to cross the site, although active traces of the Newport-Inglewood Fault have been mapped north and northwest of the subject site within the Huntington Mesa, and we believe within the alluvium of the gaps. As a result, Fault Rupture Hazard Zone (formerly Alquist-Priolo zone) for the Newport Inglewood Fault has been established by the State of California approximately 0.5 miles to the north of the site. A more detailed discussion of the Newport-Inglewood Fault is presented in Section 5.3.2 of this report.

The Southern California region is seismically active. Active and potentially active faults within Southern California are capable of producing seismic shaking at the site. It is anticipated that the site will periodically experience ground acceleration as a result of exposure to small to moderate magnitude earthquakes occurring on distant faults. Additionally, active "blind thrust faults" (faults which lack surface expression, commonly associated with fold belts and compressional deformation) or other potentially active sources (currently not zoned) may be capable of generating earthquakes. Blind thrust faults were responsible for both the 1987 Whittier Narrows (M5.9) and the 1994 Northridge (M6.7) earthquakes.

We have performed a computer aided search of the known active and potentially active faults within a 62-mile (100-km) radius of the site and we have researched the available geologic literature to determine the maximum magnitude earthquakes that may be expected to be generated on each fault (CDMG OFR 96-08). Table 1 below, summarizes 15 of the 35 known active and potentially active faults, which, in our opinion, may have the greatest impact on the site. Selection of these faults was based on the proximity of the fault to the site, and the potential of the fault to generate ground motion at the site. The site is located on the USGS Newport Beach, California 7-$^1/_2$-minute Quadrangle map, using latitude 33.656°N and longitude 117.996°W as the approximate center of the site.

Table 1 was generated using the EQFAULT for Windows computer program (Blake, 2000, Reference 2) as modified using the fault parameters from CDMG OFR 96-08. The fault distances were confirmed or revised based upon actual measurements from the "Map Showing Late Quaternary Faults and 1978-84 Seismicity of the Los Angeles Region, California" (USGS Map MF-1964) and the "Fault activity Map of California and Adjacent Areas" (CDMG Map No. 6). It is our opinion that the most significant fault that may affect the site is the Newport-Inglewood Fault Zone.



Legend:

★ Site Location

Approximate Fault Location

Earthquake Epicenters

M 5.0 to 5.9

M 6.0 to 6.9

M 7.0 to 7.9

Historic Earthquake Events

1)	1769 Los Angeles	M6?
2)	1812 Wrightwood	M7
3)	1827 Ventura County	M6
4)	1855 Los Angeles	M6?
5)	1858 San Bernardino	M6?
6)	1880 San Bernardino	M6
7)	1889 Monrovia	M5.5
8)	1893 San Fernando Valley	M5.9
9)	1894 n/o Pomona	M6
10)	1899 Cajon Pass	M6.5
11)	1899 San Jacinto	M6.4
12)	1907 San Bernardino	M6
13)	1910 Elsinore	M6
14)	1918 San Jacinto-Hemet	M6.8
15)	1920 Inglewood	M4.9
16)	1923 Loma Linda	M6
17)	1930 Santa Monica Bay	M5.2
18)	1933 Long Beach	M6.4
19)	1941 Gardena	M4.8
20)	1971 San Fernando	M6.7
21)	1973 Pt. Mugu	M5.9
22)	1979 Malibu	M5.2
23)	1981 Pt. Mugu	M6.0
24)	1987 Whittier-Narrows	M5.9
25)	1988 Pasadena	M5.0
26)	1989 Malibu	M5.0
27)	1990 Pomona	M5.3
28)	1991 Sierra Madre	M5.8
29)	1994 Northridge	M6.7

References:

Base Map:
USGS Southern California
Based on USGS 1:1,000,000
Los Angeles Map (NI-11)
(Obtained from Topo! CD-Rom)

Faults:
CDMG Map Number 6, 1994

Earthquakes Epicenters:
CDMG SP 116
Fig. 1, Page 10.

TN MN 13½°

0 5 10 15 20 25 30 35 miles
0 10 20 30 40 50 km

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ZEISER KLING Consultants, Inc.

REGIONAL FAULT AND SEISMICITY MAP

Proposed Pacific City
NEC Pacific Coast Hwy and 1st Street
HUNTINGTON BEACH, CALIFORNIA

FIGURE:	2
PN:	01039-00
DATE:	NOVEMBER 2001

By: M.W. Laney 11/09/2000 LAEQMAP1.cdr

Table 1
Significant Faults

Fault Name	Approximate Distance from Site [Miles (km)]	Maximum Event (Moment Magntitude), Mw	Fault Seismic Source Type (1997 UBC)
Newport-Inglewood (L.A. Basin)	0.6 (0.9)	6.9	B
Compton Thrust	4.3 (7.0)	6.8	B
Newport-Inglewood (Offshore)	6.5 (10.4)	6.9	B
Palos Verdes	10.0 (16.1)	7.1	B
Elysian Park Thrust	14.0 (22.5)	6.7	B
Whittier	20.6 (33.1)	6.8	B
Chino-Central Ave. (Elsinore)	22.4 (33.1)	6.7	B
Elsinore - Glen Ivy	24.8 (39.9)	6.8	B
Coronado Bank	27.0 (43.5)	7.4	B
San Jose	27.3 (44.0)	6.5	B
Verdugo	34.1 (54.9)	6.7	B
Sierra Madre	34.3 (55.2)	7.0	B
Cucamonga	35.9 (57.8)	7.0	B
Anacapa-Dume	46.2 (74.3)	7.3	B
San Andreas (Southern)	52.5 (84.5)	7.8	A

Please note that the fault distances presented in Table 1 are based on distances measured to where the fault trace is mapped or projected onto the ground surface. The distances measured from the Uniform Building Code (UBC) Near-Source Zone Maps (ICBO 1998) for use with the 1997 UBC are based on the shortest distance from the site to the fault plane projection to the surface from a depth of 10-km. Therefore, sometimes the fault distance measured from the UBC maps may be different than those presented in Table 1. The site location in relation to known active faults and historical earthwork epicenters is shown on Figure 2.

5.3.2 Newport-Inglewood Fault Zone

The Newport-Inglewood Fault Zone (NIFZ) has been fairly well studied by both structural and petroleum geologists. Like other major regional faults within the Peninsular Ranges, the NIFZ is a predominately strike-slip fault that has developed sympathetically in response to the transform-fault activity of the San Andreas Fault, which is a tectonic crustal plate boundary between the Pacific and North American plates. The fault trends roughly north-north westerly. South of Newport Beach, the fault is located offshore and trends a significant distance southerly, where it interacts with other offshore faults and possibly the Rose

Canyon fault zone of San Diego. North of Newport Beach, the fault zone extends as a series of en-echelon right-lateral faults with some degree of thrust (compressive) as far as Beverly Hills. It is terminated into the Transverse Ranges at that location, namely into the Santa Monica – Raymond Hill Fault Zones. The compressive components of this fault zone have created the structural zones of folded rock forming the oil fields, as well as topographic features of a belt of domal hills and mesas such as the Huntington Mesa, Alamitos Heights, Signal Hill, to as far north as the Domingez, Cheviot and Baldwin Hills. In all of these cases, the structure is similar- faulted domal to anitclinal features. Much of this deformation took place by late Pleistocene, although the fault remains very active today.

The right lateral stepping components of the fault zone in the near surface are "flower" expressions in response to a more singular wrench-fault feature located at depth (ie. within the actual crystalline crust). The behavior of the NIFZ overall tends to favor deformation and blind thrusting, rather than surface rupture and displacement.

This behavior explains why the large historic earthquakes on this fault zone in 1920, 1928, and 1933 had very little surface expression. No surface rupture expressions of fault movement were noted. Even in the case of oil wells directly straddling the known traces, there was little disruption.

The main effect of this fault zone on the proposed development is that of strong ground motion (shaking). This shaking may be expected and has in the past caused some degrees of liquefaction, lurching, and other secondary seismic effects, predominately limited to areas of deep soft, poorly consolidated ground with high groundwater that lacked either natural inherent strength or engineering controls to resist. Ground rupture, in the form of a surface expression of offset on the fault in an earthquake event, away from mapped existing traces, is believed remote.

5.3.3 Ground Motion

Historically, a large number of moderate earthquakes have been recorded to have occurred in the region of the project site over the past 201 years. We performed a historical search using the EQSEARCH for Windows computer program (Reference 3) from those earthquakes that are known to have occurred within 62-miles (100-km) of the site. The historical acceleration was estimated using the Boore, et al. (1997), Abramson and Silva (1997), and Campbell (1997) attenuation equations. Our search was limited to those earthquakes with magnitudes greater than M5 and through the years 1800 to 2000. A summary of the results are presented below:

Time Period (1800 through 2000):	**201 years**
Maximum Recorded Magnitude:	**M7.0 on December 16, 1858 (San Andreas Fault)**
Approximate distance to nearest historical earthquake with a magnitude greater than M5:	**3 miles, 5 km on March 14, 1933 (W on Newport Inglewood Fault)**
Maximum historic, estimated, site acceleration:	**0.4 g on March 11, 1933, M6.3 (Newport Inglewood Fault)**
Number of events exceeding a magnitude M5:	**65**

Based upon our understanding of the regional tectonic framework, the largest magnitude earthquake at the project site will most likely be generated by the Newport-Inglewood fault, with a moment magnitude of M6.9. Based on our probabilistic analysis using Blake's FRISKSP for Windows computer program (Reference 1) an acceleration of **0.45g** for alluvium (to be used as the Design Basis Earthquake) within the area may be expected to occur with a 10 percent probability of exceedance in 50 years. The site is located in Seismic Zone 4 of the 1997 Uniform Building Code (UBC). Therefore, structures should be designed in accordance with parameters given within Chapter 16 of the current Uniform Building Code.

5.3.4 Liquefaction Potential and Other Seismic Hazards

Liquefaction Potential

The alluvial soils are located in the southeastern corner of the site is located within a State of California Seismic Hazard Zone Map for Liquefaction.

From a liquefaction hazard standpoint, the site may be divided into two types of regions: Those underlain by competent natural soils (Terrace deposits), and those underlain by recent alluvium.

Liquefaction analysis of a specific, detailed nature is to be performed as a portion of forthcoming foundation specific studies.

Terrace Deposit Areas:

The majority of the site is generally underlain by terrace and engineered fill, which are in turn underlain by the terrace deposits. Based on the dense nature of the terrace and fill materials, and our analysis, the potential for liquefaction is considered to be low in these areas.

Alluvial Areas:

The southeastern corner of the site is generally underlain by loose to medium dense alluvial deposits. Based on the degree of saturation observed in these areas in the borings and cone penetration test data from the previous reports, the relative densities of the soils observed, it is our opinion that the potential for liquefaction within the alluvial area is moderate to high, were left unimproved.

Seismically Induced Slope Failures

The site is not located within a State of California designated Seismic Hazard Zone Map for Slope Stability. Since there are no significant slopes within the site boundary, the potential for seismically induced slope instability is considered low to remote.

Tsunami and Seiches

With respect to tsunami, the site is located within an area of "moderate" tsunami run-up as defined in Figure EH-8 of the City of Huntington Beach General Plan. According to Figure 208 of USGS Professional Paper 1360, the potential for up to 8-feet of tsunami wave run-up may be expected during a 500-year seismic event.

Due to the lack of land-locked bodies of water (i.e. ponds or lakes), the potential for seiche is considered to be non-existent.

Other Seismic Hazards

Risk of ground lurching, cracking or seismically induced spreading or compaction effects were also evaluated. The geologic units are dense to overconsolidated terrace alluvium, and medium dense alluvium. The potential for ground lurching, cracking or seismically induced spreading or compaction effects within these areas are considered low, especially considering the engineering controls and corrective grading anticipated to be performed for the proposed development.

The primary geoseismic risk anticipated at the site is that of strong ground motion as a result of activity on distant faults, as described already in Section 5.2 and summarized in Table 1 above.

5.4 Groundwater

The geologic and hydrogeologic aspects of groundwater at the site are discussed in the Section 5.1 above.

Groundwater at the site was encountered at depths of approximately 5- to 24-feet below the ground surface (bgs) during our site investigation, which corresponds to approximate elevations of –1/2 feet below to 4 feet above mean sea level (MSL). On July 30, 2001 the piezometers (ZB-2, ZB-5 and ZB-5A) were measured, and the groundwater levels were found to be between approximately 9- and 20-feet bgs (approximately 3- to 4-feet above MSL).

Based on the past use of the site, the groundwater may be contaminated. Our understanding is that the current remediation performed by the previous landowner should limit such contamination to nuisance levels or below. If dewatering is required during construction, environmental testing and remediation of the discharge water should be incorporated into the disposal plan.

5.5 Settlement Potential

The southeastern portion of the site is underlain by approximately 15- to 20-feet of settlement prone alluvial/lagoonal deposits. Under currently proposed fill loads, settlement of these soils could be on the order of ½-inch for each foot of fill placed over a period of several months. Building loads imposed on settlement prone soils will increase both the magnitude and duration of settlement to occur. Settlement due to primary and secondary consolidation under typical foundation loading could cause structural and service related distress to structures in this area without mitigation of settlement prone soils.

The site is not within a area that has been impacted by long-term subsidence due to local oil extraction according to the Huntington Beach General Plan.

The settlement potential of the buildings should be performed on a case-by-case basis once for finalized plans are produced.

5.6 Other Hazards

The project site has the following additional issues that can impact the construction and development of the site:

5.6.1 Abandoned Oil Wells and Methane

The site is located within a former Chevron oil field. As mentioned in Sections 2.1 and 2.2, several abandoned oil wells exists within the site.

Additionally, according to Figure EH-10 of the 1996 Huntington Beach General Plan, the site is located within a Methane Overlay District. This condition requires a review by the Huntington Beach Fire Department (City Specification 429) which requires a site soils testing plan to determine the presence of methane gas and/or soil contamination. It is our understanding that this study was performed by Harding Lawson ESE, Inc. as discussed in their January 31, 2001 Remediation Plan, and the recommended remedial grading to be implemented by Chevron and their representatives.

A copy of the Huntington Beach City Specifications 429 and 431-92 are included as Appendix F.

5.6.2 Ocean Related Corrosion Potential

The site is located approximately 500-feet from the Pacific Ocean. Building materials, such as metal, stucco, plastics and others are prone to corrosion and deterioration due to the presence of salts in the air and humidity from the evaporation from the ocean. Therefore, the ocean breezes and winds that will be blowing across the site should be considered to be corrosive towards metals and concrete, and the architect should take these conditions into consideration when assigned building materials for the proposed structures.

5.6.3 Flood Hazards, Storm Surge and Transient Groundwater

According to Figure EH-11 of the 1996 Huntington Beach General Plan, the majority of the site, the northwestern two-thirds, is located within an area of minimal flooding (Zone X, 500-year flood according to FEMA). According to the General Plan, the southeastern third of the site is located within an area that can be flooded from 1- to 3-feet in the 100-year event (Zone A99, Special High

Risk Flood Area, according to FEMA). The site is also on the border of the area that may experience flooding due to wave action, according to the General Plan.

Storm surge is a phenomenon that occurs primarily during severe storm events. It is a rise above normal water levels along a coastline due to the action of wind stress on the water surface. Since the site is located approximately 500 feet from the ocean and due to the lack of hurricane like storm conditions in this region, the potential for the site to be impacted by surging is low.

The site groundwater may be impacted by rises in the ocean tides, water infiltration during heavy storm events and surrounding irrigation, resulting in a transient groundwater condition. The building foundations and slabs should be designed to accommodate temporary rises of the groundwater table.

6.0 CONCLUSIONS

The results of our study have concluded the following:

- The existing site and vicinity is considered absent of any geotechnical conditions that would preclude the proposed construction. The site does, however, include geologic, geotechnical and hydrogeologic conditions that influence the constructability and long-term performance of the development. This firm and the design team are addressing these conditions and specific protocols and recommendations have or will be developed for these aspects.

- The proposed development is considered feasible, however, remedial grading, deepened foundations, or ground improvement methods as described in this report, will be required to limit adverse settlement, and earthquake related distress.

- The proposed development is not anticipated to adversely affect adjacent properties from a geotechnical viewpoint, provided the proposed grading and construction incorporates the recommendations of this firm.

- Due to the presence of relatively shallow groundwater, excavations deeper than an elevation of about 9 feet above MSL will most likely require specialized excavation methods.

- The southeastern portion of the site consists of compressible "gap terrain" soils, and will require remedial grading, deep foundations, ground improvement or a combination of these techniques depending on the final grading plans and proposed structure loading. Conceptual recommendations for planning purposes are provided in this report.

- The northwestern portion of the site is underlain by terrace deposit material, and should provide adequate bearing characteristics for currently proposed foundations. However, based on the height and weight of the proposed structures, mat or deep foundations may be required. The individual building foundations can be evaluated on a case-by-case basis once plans are available.

- The site is located within a Tsunami Run-Up Zone according to the City of Huntington Beach General Plan. This could have an effect on proposed structures within low-lying areas of the site.

- The site is located within a Methane Hazard Zone according to the City of Huntington Beach General Plan. Methane testing will need to be conducted, and remediated in accordance with City Specification No. 429, in accordance with City of Huntington Beach Fire Department requirements.

- Based on the sites past use as an oil field, the potential for exposure to hazardous waste in the soil and groundwater is possible. It is our understanding that the owner is aware of this issue and that remediation and work plans are in progress to address it.

Our conclusions as stated above are based largely on the understanding that this firm will be retained to observe, test, and comment on the earthwork and construction. Some as-grading, depending on conditions actually exposed at the time of earthwork, is also anticipated. Additional studies should be performed by this office to evaluate specific conditions and detailed geotechnical aspects of the site and proposed development once comprehensive and smaller scale (i.e. 100-scale or 50-scale) grading and design plans are formulated. An As-Graded report should be prepared to document the nature of the actual grading performed.

6.1 Local Engineering Geologic Considerations

The following engineering geologic considerations regarding the site conditions should be considered in the grading plan design:

- The groundwater table is approximately at elevations of –8 to 4 feet below/above MSL. If bottom of spread foundations and/or slabs-on-grade are below an elevation of 9 feet MSL, than dewatering will most likely be required prior to construction.

- The alluvial soils in the southeastern third of the site are highly compressible, and are subject to significant settlements under structural loads. Therefore, it is our opinion that this area of the site is unsuitable for use of conventional foundations and slabs-on-grade.

- Within the portions of the site underlain by Terrace, overexcavation and recompaction of the soil beneath the proposed structures should be acceptable for lightly loaded structures. Some questionable zones may exist within the terrace materials with respect to "heavy" capacity, and deep foundation members will need to be extended through them, where they exists. For this and other reasons, foundation specific detailed studies will need to be performed.

- Between the elevations of –30 and –40 feet MSL in the southeastern third of the site exists a very dense sand layer that is well suited for driven piles or drilled shaft caissons (constructed with casing or slurry methods) for multistoried structures.

- Terrace Deposits are generally overconsolidated, and have abundant cohesive soil components. Terrace Deposits represent the most likely bearing materials for the proposed development. Although shallow foundations may be feasible for light to moderately loaded structures, multi-storied buildings may require deep foundations, such as driven piles or drilled shaft caissons, or mat foundations, due to heavy structural loads. Special, detailed foundation specific studies for each building is recommended.

- No active faults have been observed on the site to date. Additionally, the southeastern portion has a moderate potential to liquefy during the design level earthquake, with an estimated settlement of less than an inch.

- A majority of the on-site, near surface soils, exhibits a medium to high potential for expansion. However, if the recommendations presented by this firm are followed, these effects can be minimized.

- Sulfate resistant concrete will be required to mitigate the corrosive effects of the on-site soils. Additionally, underground utilities will also need to be protected from corrosive soils relative to both concrete and metals.

7.0 PRELIMINARY DESIGN CONSIDERATIONS

This section discusses the geotechnical recommendations with respect to the proposed construction and grading. General Earthwork Specifications for typical grading procedures are presented in Appendix H. The recommendations presented in the body of this report supercede any contradicting recommendations presented in Appendix H. These general specifications are intended only to supplement the specific recommendations discussed below and elsewhere in this report.

Note: The following recommendations are preliminary only, and will be supplemented with forthcoming, site and building specific studies and grading plan reviews. Once more detailed information on the site conditions are known, we would be able to provide more refined design recommendations.

7.1 Demolition and Unsuitable Materials

As described in the body of the report, the site includes slabs, foundations and asphalt pavement from the previous motel and restaurant buildings. These features will need to be demolished. It is anticipated that the main structural portions of these structures would be removed in the process of clearing and grubbing the site. Any existing utilities will also need to be abandoned and also removed or otherwise suitably demolished. The debris should be removed from the site. This firm should document these operations.

Vegetation, particularly trees and shrubs will need to be grubbed and removed from the site as well. The vegetation will need to have the major aspects of their root structure removed, and care should be taken to limit the amount of roots and other organic material that remain in the ground that could be incorporated into the fills. The resulting debris should be removed from the site. This office should also document these operations.

The desired degree of demolition and grubbing should result in a surface within the grading limits that is essentially free of objectionable or otherwise deleterious materials, and is adequately cleared to allow for unrestricted earthwork to commence. It is anticipated that some foundation remnants and localized areas of underground utilities will remain after the demolition process. Based on our experience with similar sites, it is believed that these remnants will be exposed, demolished, and removed in the process of the recommended overexcavation of the site. These features, provided that the construction debris is of a non-consolable nature, can be suitably broken up and dispersed, and may be incorporated into the replacement fills on a limited basis. This should be evaluated on a case by case basis by the geotechnical consultant at the time of grading. Otherwise, they should be culled and disposed of offsite.

Organic remnants, such as deeper roots from large trees, are not considered suitable for incorporation into the fill and should be removed from the underlying natural ground. Therefore, such materials will need to be chased out by local overexcavation to the satisfaction of the geotechnical consultant in the field at the time the removals are made. The roots and other materials should be extracted from fill soils and removed from the site.

Although evaluation of hazardous wastes is beyond the scope of this study, based on our current understanding it is anticipated that environmental hazards will most likely be encountered in the course of demolition and grubbing. It is our understanding that the remediation and disposal of contaminated materials is to be performed under the observation of an environmental engineering firm.

Geotechnical clearance from this office is recommended with respect to the adequacy of the demolition and grubbing operations prior to the commencement of actual grading. This is in addition to any jurisdictional clearances required.

7.2 Soil Removals

The soils in the upper several inches to a foot over most of the site are generally topsoil-like in nature, and are organic enriched. As such, they are considered unsuitable. These soils may be dispersed into the planned fills provided the large or obvious concentrations of organic material are removed and the soils do not contain more than 5% organic debris. Any organic rich soil allowed will need to be processed and blended into the mass fills to the satisfaction of the geotechnical consultant in the field and such that the amount of total organics does not exceed 2% in any portion of the final fills. If this cannot be accomplished, these soils should not be incorporated into the engineered fill prisms, and will require disposal.

Organic rich soils may be of value, however, with regard to post-grade landscape uses. It may be desirable to stockpile these soils for such use.

The southern areas of the site underlain by "gap" alluvial soils will be influenced and limited to some extent by groundwater. Conceptual methods of addressing these removals are presented herein. Specific protocols and recommendations will be presented as a portion of our forthcoming building/foundation specific reports, and the grading/foundation plan reviews.

7.3 Overexcavations in Natural Areas

Due to weathering and disturbance, the surficial soils within the limits of grading are generally not considered to be adequate for support of new fills and structures in their current conditions. These soils may be overexcavated, processed, and replaced as engineered fills to provide the required support. The effect of the overexcavation would be to provide a uniform, controlled subgrade for the support of the proposed additional fills and foundations. A detail for the overexcavation is included in Appendix H.

The current conceptual general overexcavation recommendations are:

- Remove upper 10-feet of natural ground as measured from the natural ground surface in the alluvium (Qal) in the southeastern portion of the site, and the upper 5-feet in the terrace deposits (Qt) where engineered fills are proposed to be less than or equal to 15-feet higher than original grade. This will most likely require dewatering in the lagoonal/alluvial areas, and the installation of a slurry cut-off wall along PCH.

- Where design fills will exceed 10-feet from the previous existing grades, the depth of overexcavation in alluvium (Qal), as a general rule, should be one-half of the difference of the finished grade elevation and the existing natural grade elevation, or to terrace. Within the terrace deposits, the depth of overexcavation should be one-quarter of the difference of the finished grade elevation and the existing natural grade elevation, or to competent terrace.

  The overexcations may extended below the groundwater table. This may require dewatering, and/or chemical treatment (i.e. lime or cement treating) of the subgrade soils to facilitate the removals. Special recommendations for such will be presented in forthcoming reports.

The overexcavation bottoms should be mapped and evaluated by the project engineering geologist. If, in the opinion of the geotechnical consultant, adverse conditions exist, these conditions should be removed or otherwise mitigated to the satisfaction of the consultant. Fills should not be placed until the overexcavation bottom has been observed, evaluated, and approved by the geotechnical consultant.

Once the overexcavation has been adequately accomplished, the bottom should be prepared to receive fill by scarification and moisture conditioning, or as otherwise recommended by the geotechnical consultant.

The exposed bottoms in the southeastern portion of the site may expose saturated zones, making a competent bottom difficult to attain. If this condition is encountered, the bottom may be stabilized as discussed below.

Unstable Excavation Bottom Above Groundwater Table

In areas were the exposed bottom is saturated but above the groundwater free surface and a competent bottom is not attainable, we recommend that the bottom be stabilized. One method of stabilization is through the use of crushed rock and a geotextile fabric. We estimate that on the order of 2- to 4-feet of rock may be necessary to "bridge" the bottom, however, this thickness will need to be considered on a case by case basis. A representative of our firm should monitor the placement of this rock to evaluate its effectiveness. The geotextile should be non-woven (such as Mirafi 600X, or equivalent) and should be placed directly on the overexcavated subgrade according to the manufacturers recommendations. The fabric should be placed in the direction that the rock is to be placed and adjacent panels should overlap a minimum of 2-feet. The fabric should be draped on the edges of the excavation so that the fabric will encapsulate the sides of the rock layer. If the fabric is damaged during installation, the damaged area should be exposed and a patch of fabric should be placed over the area that extends at least 3 feet beyond the damaged area. Vehicles or earthwork equipment should not drive directly on the fabric.

The rock may consist of ¾-inch crushed rock, placed in lifts of no more than 12- to 18-inches and densified in place. Care must be taken to avoid overstressing the subgrade and minimize repeated agitation of the subgrade soils until a stable bottom is achieved.

Following placement of the crushed rock, the rock layer should be covered with another layer of the geotextile fabric. The upper layer of fabric will serve to reduce the migration of fines into the rock from the compacted fill. The upper layer of fabric should be generally placed under the same guidelines as those for the bottom layer of fabric. Engineered fill should be placed on the fabric, in such a manner so as not to damage the fabric, and compacted as discussed in Section 8.3.5 of this report.

Unstable Excavation Bottom At or Below Groundwater Table

If the excavation is at or below the groundwater table, special considerations to dewatering and the installation of a slurry or sheet pile cut-off wall will need to be considered. Additionally, the subgrade will need to be stabilized using chemical treatment (i.e. cement or lime treatment). This issue is to be studied in more detail in subsequent studies.

7.4 Oversized Material

In general, rocks and other hard irreducible particles should be limited to 12-inches or less in nominal diameter. Particles larger than this should be culled from the fills (and may be stockpiled for landscape uses if desired). Smaller oversize particles may be dispersed through the fill under the observation, testing and documentation of this firm, on a case by case basis. The upper 3-feet of the final subgrade should remain free of oversize particles to limit interference with proposed structures, utilities, and landscaping.

7.5 Preliminary Foundation Parameters

Preliminary foundation parameters are provided below based on our recent exploration and exploration conducted by previous consultants on the subject sites. Since proposed building loads and site grading are unknown at this time, foundation parameters presented below should be considered preliminary, and be used for planning purposes only.

Note: Once proposed building loads and proposed site grades are known, and additional building-specific geotechnical investigations are completed, preliminary foundation parameters presented below should be revised in lieu of more recent, site and building specific studies.

7.5.1 Shallow Foundations

Proposed lightly loaded, low-rise structures (less than 4 stories) may be supported on typical spread and mat foundations. Based on our preliminary testing and our understanding of the subsurface conditions, we have developed the following preliminary foundation parameters for shallow foundations:

Bearing Capacity	2500 psf
Friction Coefficient	0.35
Passive Resistance	300 psf/ft
Minimum foundation depth	5.0 feet
Minimum Foundation Width (continuous footings)	2.0 feet

7.5.2 Deep Foundations

We have divided the site into two areas for deep foundation design. Figure G-1 in Appendix G illustrates the Pile Design Areas to be used with the Preliminary Pile Design Charts.

Structures proposed in the southeastern third of the site, Area 1, should utilize deep foundations bearing into dense terrace deposits, however, additional foundation depth will most likely be required to encounter competent soils, and counteract potential negative skin friction due to settlement of surrounding soils. Figure G-2 in Appendix G provides preliminary allowable downward and uplift design loads for driven piles. Due to the nature of these soils and depth to groundwater, cast-in-drilled hole (CIDH) piles are not recommended.

Mid-rise structures (greater than 4 stories, including subterranean parking) in Area 2, may also be supported on deep foundations bearing into dense terrace materials encountered at approximately 30 to 40 feet below grade. Figures G-3 and G-4 in Appendix G provides preliminary allowable downward and uplift design loads for CIDH and driven piles, respectively. If CIDH piles are to be considered, than casing and/or drilling mud stabilization methods of construction will need to be considered in the cost of the project.

Preliminary parameters for deep foundations have been provided in Appendix G. These parameters should be considered for preliminary planning purposes only, to be verified and supplemented by site-specific studies once building loads and configurations are known.

7.6 Expansive and Corrosive Soils

Expansive Soils

As discussed in the laboratory testing section and described elsewhere in the report, the soils anticipated to be exposed at grade in contact with the foundations proposed are expected to have an medium to high expansive potential. Portions of the site, as tested, vary significantly with respect to clay content and expansion potential. For this reason, testing of the near surface soils exposed at pad grade should be performed to determine actual expansion index values and Atterberg Limits, and the foundation designs modified accordingly if necessary after rough grading.

For the purposes of this report and development of preliminary foundation plans, we assumed the soils to be highly expansive.

Corrosive Soils

Based on our laboratory testing, the soils to be exposed at or near pad grades are expected to be severely corrosive to both concrete and buried metal. The brackish nature of portions of the on-site groundwater should also be considered in evaluation of the corrosion potential of the on-site soils. During grading, testing of the near surface soils exposed at pad grades should be performed to determine actual corrosivity values, and the foundation designs modified accordingly if necessary. Concrete mix design should preliminarily assume "Severe" soluble sulfate exposure in accordance with 1997 UBC Table 19-A-4.

We recommend that a qualified corrosion engineer be consulted to develop recommendations for cathodic protection or other means to limit corrosion of buried metal on the subject site.

Retaining Wall Considerations

Retaining walls should be composed of materials resistant to the corrosive effects of the soil. Mechanically Stabilized Earth (MSE) or other non-conventional retaining structures should have manufacturer provided data regarding the corrosion resistance of these products. Conventional walls should be constructed using Sulfate Resistant Concrete in accordance with Table 19-A-4 of the 1997 Uniform Building Code. See Section 10.7.2 "Concrete" for specific recommendations.

Expansive soils can exert a very high stress into a conventional retaining wall system if allowed. To insulate the wall from these effects, a zone of drained, granular material should be provided within a 2/3:1 projection of the back toe of the wall. To protect the wall components, the low expansion backfill should be free of cobbles and boulders. The wall should also be waterproofed.

The low expansion material should be relatively free draining and have an expansion index of 20 or less. this low expansion material should be mechanically compacted to 90-percent relative compaction.

A detail illustrating the retaining wall backfill recommendations is presented in Appendix I.

Vapor Transmission

Unprepared concrete, masonry, and similar materials in direct contact with the subgrade soils may be expected to transmit or wick water vapor and even liquid through capillarity when in contact with damp ground. This wicking effect may also transmit soluble salts dissolved in the water through the concrete. This may result in objectionable salt deposits and moisture condensate. The effects may be limited by a using high strength and low water-cement ratio concrete mix, and by use of an appropriate vapor barrier. The underdrainage recommended herein should also serve to limit such transmission.

Due to the potential presence of methane at the site, we recommend that a qualified environmental engineer review the potential for methane to be present within the subsurface, and provide recommendations for mitigation of high methane concentrations.

7.7 Subdrainage and Underdrainage

The site will include two primary types of subdrains:

1) Backdrains for retaining walls.
2) Underdrainage for the Hotel and Entertainment Center

Retaining Wall Drains

The retaining wall drain details have been described in other portions of this report and are also presented on the attached detail in the Appendix I. The installation of these drains and their outletting should be under the observation and testing of the geotechnical consultant. The retaining wall drains should be outletted into an appropriate drainage system.

Where moisture flux through the wall is undesirable, the back of the wall below grade should be waterproofed with a suitable waterproofing. A 3-part bituthane waterproofing involving a primer/sealer, self-adhesive flexible membrane, and protection board (i.e. Miradri or equivalent) should be considered.

Drainage behind the wall may consist of a conventional perforated drainpipe encased in gravel and wrapped in filter fabric (as per detail in Appendix I). As an alternative, drainage may also be provided for behind the wall using a geosynthetic drainboard system which is tied into a collector pipe and outletted (i.e. Miradrain, Quickdrain, or approved equivalent). If such an alternative is desired, details and recommendations will be provided separately.

Underdrainage for the Hotel and Entertainment Center

As already discussed in this report, the site is situated in an area of relatively shallow groundwater. Many of the proposed basement foundation elevations will be located very near the current groundwater level.

The groundwater is anticipated to remain relatively stable, given the current structure in which it is situated, in the long term. However, as already discussed, transient fluctuations due to exceptional climatic events, or changes in use and influx may induce changes in the groundwater table. The most likely scenario affecting the proposed development in the foreseeable future, based on our understanding, is transient fluctuations of short duration that may result in a temporary rise in groundwater.

Although it is strongly anticipated that the construction of the subterranean/basement foundation portions of the development will include relatively "watertight" construction materials and methods, it is inevitable that nuisance water and moisture transmission may occur. The occurrence of these nuisance influxes of water and moisture would be both as direct capillarity and wicking, as well as transmission through any flaws, joints, cracks or similar features.

The magnitude of such nuisance transmissions are expected to be greatly exacerbated should groundwater impart an increased head pressure. Because of a lack of freeboard between the basement foundations and the groundwater, even small transient groundwater elevation changes may affect the performance of the basements if no controls are provided.

For these reasons, it is recommended that an underdrainage array be installed under the base slab foundations in addition to the other waterproofing controls planned. This is particularly true for the entertainment/hotel facility planned on the northwest portion of the site fronting PCH.

The underdrainage array is essentially a permeable sheet drain of either gravel or composite drain construction that application has a manifold system of pipes that may transmit the transient groundwater, when it occurs, to a suitable discharge. These drains would only be active during periods of high groundwater, and then resume a passive condition forming a capillary break against vapor/moisture when the groundwater levels return to normal.

Depending on the method of overexcavation and bottom stabilization utilized, the gravel bedding and geofabric used for that may be incorporated into the drain system. Further, the underdrainage may be at least partially installed in the excavation phase to bleed off seepages in the open excavation.

The underdrainage and the retaining wall backdrainage may be incorporated together, provided the incorporation limits cross transmission between the two systems. Specific recommendations and details of the underdrain systems would be included within future foundation-specific studies and grading plan review.

7.8 Site Protection

The site should be protected by appropriate erosion control, especially during wet weather periods. When grading is to be performed in inclement weather, it is the contractors responsibility to leave intermediate grades that allow for collection and removal of runoff and minimize damage potential. Any damage from such inclement weather effects should be repaired to the satisfaction of the geotechnical consultant based on actual field conditions.

Similarly, the site should not be allowed to "dry out" or allow the pad surfaces to desiccate. This may be prevented by regularly sprinkling the pad surfaces to maintain the recommended moisture content. Should the pads dry out from exposure, they should be adequately rehydrated and reprocessed to the satisfaction of the geotechnical consultant prior to the excavation of foundations.

Foundation and other temporary excavations should be appropriately protected. Slough and debris should not be allowed to fill into foundation excavations. Common sense and OSHA guidelines should be employed with respect to excavations onsite.

Slopes should be protected until vegetated and the vegetation has become established. To assist in slope performance, a jute net and hydromulch system may be appropriate. Specific recommendations for such would be provided both in subsequent grading plan reviews and on a case by case basis during grading.

7.9 Surface Drainage

The long-term performance of the site and structures will be significantly enhanced by attention to providing and maintaining proper surface drainage.

Surface drainage in the form of appropriate grades and capture devices should be provided for. It is recommended that roofs be guttered and drained. The roof downspouts should be outletted into an area drain system. Similarly, hardscape should also be provided with area drainage.

Planters around the proposed buildings should be avoided where possible. Planters and landscape areas should be provided with adequate drainage. This drainage should be maintained, and adjusted accordingly if the landscaping is altered.

The soil grade within 5-feet of the proposed foundation should be sloped to drain 2% or greater away from the foundations. The drainage should be directed to an appropriate area drain system. A 1-foot berm should be provided for the top of slope, and pad drainage should be directed away from the tops of slopes. Drainage water should not be allowed to flow uncontrolled over any slope.

7.10 Settlement Potential

On-site alluvial soils, particularly in the southeastern portion of the site will be subject to settlement beneath proposed building and fill loads. Settlement of these soils could cause structural and service related distress.

These settlement concerns will be addressed on a site/building specific basis as a part of our forthcoming foundation specific studies and grading/foundation plan reviews.

7.11 Ground Improvement

In order to reduce potential settlement, decrease liquefaction potential, increase shallow and deep foundation bearing capacity, ground improvement techniques may allow for lower construction costs, and shorten the project schedule the following options for ground improvement are being presented. Engineered ground improvement would have a particularly pronounced effect in the southeastern portion of the site. Ground improvement techniques are described below:

Stone Columns
Stone columns typically consist of a column of granular material, usually open-graded gravel, which replaces settlement prone or poor permeability soils which serves to enhance drainage and reduce settlement time. Stone columns can be placed by drilling a large diameter hole in which the gravelly material is placed, or the gravelly material can be placed with a "vibro-replacement" technique. Vibro-replacement has the advantage of densifying surrounding soils with the use of a vibratory probe, and replacing the resulting void with granular materials. Vibro-replacement also works well with settlement prone soils below the groundwater level.

Grouting
Grouting typically involves injection of a stiff soil-cement mixture under pressure within subsurface soils to displace and densify compressible soil materials. Grouting can also include injection of fluids to permeate subsurface soils, or specific lenses of material can be treated to limit potential settlements.

Surcharging
Surcharging consists of placing a temporary overburden load over compressible soils materials in order to densify in-place soils prior to placement of foundations or other improvements on the site. The time required to complete settlement under a surcharge load may be decreased if drainage layers consisting of granular material or synthetic "wick drains" are incorporated into the compressible material.

Soil Treatment/Stabilization

The above-mentioned methods are considered effective, but potentially costly and time consuming. Given the current design scheme and development layout, it is our opinion that a nominal overexcavation with chemical treatment (such as lime or cement treating) combined with geogrids may be the most cost effective method for reducing differential settlement and providing a stable bottom for the proposed slab-on-grades.

7.12 Tsunami Protection

The City of Huntington Beach general plan indicates that a probable Tsunami will entail a run-up of approximately 8 feet above sea level. Living and other occupied spaces should be located a minimum of 8 feet above mean sea level, with an adequate freeboard to be determined by the project coastal engineer.

8.0 ROUGH GRADING RECOMMENDATIONS

The following recommendations are provided for preliminary planning. They are subject to refinement and augmentation based on studies focused on the review of detailed grading and building plans, once they are formulated.

These recommendations are for preliminary purposes only and will be superceeded by further studies.

8.1 Site Preparation

The site should be prepared for grading by incorporating the recommendations of this report and those to be presented in the field.

8.2 General Grading Requirements

General Grading guidelines and requirements are presented in Appendix H. The guidelines are to be supplemented by further grading plan reviews and input from the geotechnical consultant based on the actual conditions exposed.

8.3 Special Grading Considerations

Groundwater is a consideration where removals will be excavated to within a few feet of sea level.

8.3.1 Demolition of Existing Structures

The existing on-site structure remnants are to be removed, along with their foundations and the debris hauled offsite. These procedures are described in Section 7 of this report.

8.3.2 Vegetation Removal and Grubbing

Vegetation within the grading limits should be removed, the roots grubbed, and the debris removed from the site. The procedures are described in Section 7 of this report.

8.3.3 Excavation Difficulty

The terrace, alluvial and fill materials are considered generally rippable with conventional earthwork equipment.

8.3.4 Dewatering

Dewatering of the majority of the proposed excavations is not anticipated to be necessary. However, the required removals in the southern portion of the site may need dewatering. Most likely a slurry cut-off wall may be required along PCH to reduce the amount of groundwater flow into these excavations. This will require further investigation during the site-specific study for the proposed hotel and entertainment center.

8.3.5 Engineered Fills

Engineered fills are to be placed in approved removal bottoms in order to achieve design grade. These soils will comprise the direct major foundation soil for the development.

It is recommended that the engineered fills at the site be placed at 90-percent or better relative compaction. The soils should be moisture conditioned to a moisture content of 120% of optimum or higher.

The engineered fills should be placed and processed in thin, horizontal lifts such that the specified compaction may be attained. Each lift should be compacted to a uniform and unyielding condition. The actual placement, processing, and compaction should be performed under the observation and testing of the geotechnical consultant.

Should areas of less than the required compaction and placement specification be encountered, the affected area should be reprocessed or removed and replaced, such that the required conditions of compaction, moisture and mixing are obtained.

All engineered fills are to be placed to the specifications presented in this report and subsequent reports of this firm, as well as to the satisfaction of the geotechnical consultant in the field, and the requirements of the City of Huntington Beach.

8.3.6 Bulking and Shrinkage Values

The existing fill, alluvial and terrace soils to be overexcavated and utilized as a base stock for the proposed fills are variable with respect to potential for bulking and shrinkage. Existing fills, topsoil and lagoonal deposits are expected to have the highest relative shrinkage potential when compacted. The deeper portions of the terrace within the limits of overexcavation are, conversely, anticipated to bulk somewhat. The net value will depend strongly on the depth of the overexcavation section, and influence of removal of any oversize or losses from soil incorporation in removed demolition and grubbing debris.

For working purposes, the following may be assumed:

- Topsoil and Fills: Shrinkage of 10% to 15% net.
- Lagoonal Alluvial Deposits: Shrinkage of 10% to 15% net.
- Upper 5-feet (from existing grade) of Terrace: Shrinkage of 5% to 10%.
- Lower 5 or more feet of Terrace: Bulking of 5% to 10%.

These values are preliminary only, and will be refined in subsequent field studies.

8.3.7 Inspection of Temporary Slopes and Overexcavations

The excavations for the above-described features should be performed under the observation and testing of the geotechnical consultant, as described in this report. The excavations are subject to evaluation and acceptance by this firm based on exposed conditions, and ultimately, by the City of Huntington Beach. Should unsatisfactory conditions be encountered, the conditions should be corrected to the satisfaction of the geotechnical consultant, and any changes to the anticipated design noted and reported in the as-graded report.

The excavation limits and bottoms should be appropriately surveyed to document removal limits and allow for a crosscheck of quantities removed.

No fills are to be placed, nor is the bottom to be processed unless the excavation had been approved by the geotechnical consultant and the City of Huntington Beach.

8.3.8 Subdrain, Backdrain and Underdrain Installation and Inspection

As described in the applicable sections of this report, subdrains and underdrain devices should be placed and outletted under the observation, testing, and documentation of the geotechnical consultant. The location, limits and elevations of the drain systems should be surveyed. The results should be presented in the as-graded report.

These devices are subject to geotechnical release and acceptance by the City of Huntington Beach. No fills are to be placed over the drain until the subject drain has been released by the geotechnical consultant and the City of Huntington Beach.

8.3.9 Pad Construction

Pads are to be constructed in accordance with the detailed recommendations presented in this and subsequent reports, and to the satisfaction of the geotechnical consultant in the field. The pads are to be compacted to the recommended moisture and 90-percent relative compaction to the surface of the pad. Finish testing of each pad is recommended. The pads, when finished, should include appropriate drainage, per City of Huntington Beach / County of Orange requirements.

8.3.10 Existing Utilities

Existing utilities are known or are thought to be located on site. Many within the main portion of the site are associated with the previous oil field operations or the former businesses and are considered abandoned. Where such utilities of an abandoned nature are encountered, they should be properly removed or remediated. This may be done on a case by case basis during grading.

Existing utilities, namely water, as well as other lines, are known to exist along PCH. Many of these utilities are of a main or "trunk" nature and are active. It is our understanding that there is a 18-inch water main running through the site parallel to PCH.

Where such lines cross or lie within proposed building areas, they will require relocation, preferably outside the grading (overexcavation) limits.

Many of the utilities within the southeastern portion of the site are anticipated to be influenced by any proposed fill that may be planned. It is recommended that a detailed study of these influences be made once appropriate plans for the project are formulated, and appropriate remediation be implemented to protect and/or relocate these utilities.

8.4 Pre-Grade Meeting

A pregrade meeting is recommended prior to the commencement of grading operations to discuss the recommendations of this report, clarify the schedule and approach, and to introduce the key personnel associated with the project work.

8.5 Observation and Testing in Construction

The subject earthwork should be performed and documented under the observation and testing of a geotechnical consultant in accordance with the standard of practice, as well as the requirements of the City of Huntington Beach and the County of Orange. The recommendations made in this report are based largely on the assumption that this firm will be retained to provide full time observation and testing of the subject earthwork. Should this not be the case, the recommendations and advice presented may be considered void unless adopted in full or in part, or suitably revised as deemed necessary by the succeeding geotechnical consultant.

The results of the observation and testing documentation, field mapping and any as-graded changes made to accommodate actual conditions should be presented in an As-Graded report at the completion of grading.

9.0 POST GRADING RECOMMENDATIONS

The following recommendations are provided for preliminary planning. They are subject to refinement and augmentation based on studies focused on the review of detailed grading and building plans, once they are formulated.

9.1 Seismic Design

Based on the soil conditions encountered at the site and the 1997 UBC, along with the maps of Known Active Faults Near-Source Zones in California and Adjacent Portions of Nevada (1998), the following seismic design parameters may be used for the site.

Seismic Parameter	Recommended Value (Soil)	Recommended Value (Bedrock)
Seismic Zone Factor, Z	0.40	
Governing Fault	Newport-Inglewood Fault (L.A. Basin)	
Seismic Source Type	B	
Distance to Site	< 2 km	
Soil Profile Type	S_D	
Seismic Coefficient (C_a)	0.44 N_a	
Seismic Coefficient (Cv)	0.64 N_v	
Near Source Factor(N_a)	1.3	
Near Source Factor(Nv)	1.6	

Site specific Response Spectra will be provided for the mid-rise structures in future building specific reports.

9.2 Pad Preparation

The building pad should be cleared of vegetation, or any deleterious materials prior to construction. Disturbed or loose soils should be removed, moisture conditioned, and recompacted to at least 90 percent of the laboratory maximum dry density.

9.3 Conventional Foundation

For structures supported by conventional shallow spread and continuous footings, an allowable bearing pressure of 1500 pounds per square foot is recommended for footings having a minimum width of 24-inches and a minimum depth of 24-inches below the lowest adjacent grade. Spread footings should be at least 24-inches wide, founded at least 24-inches below the lowest adjacent grade. These values may be increased by 250 pounds per square foot for each additional foot of width or depth to a maximum value of 2500 pounds per square foot. If normal code requirements are used for seismic design, the allowable bearing value may be increased by one-third for short duration loads such as the effect of wind or seismic forces.

A friction coefficient of 0.35 between soil and concrete may also be used for design. For calculating passive pressure, an equivalent fluid weight of 250 pounds per square foot per of depth may be used. However, when combining both passive and frictional resistance, one or the other should be reduced by one-half.

9.4 Deep Foundations

Deep foundations, consisting of either driven, concrete piles or cast-in-drilled hole (CIDH) piles will most likely be required for the multi-storied structures on site. Specific recommendations for the indicator pile program and construction/installation of the driven and CIDH piles will be included in the future site specific studies.

9.5 Concrete Slab-on-Ground

Slabs with floor coverings should be underlain by a 6-mil visqueen moisture retarder with a two-inch layer of sand over the visqueen and a two-inch layer of sand (nominal) below the visqueen. For slab conditions not condusive to such a system, alternative recommendations will be provided for at the request of the client. Should vinyl, wood or other highly sensitive to moisture floor coverings be contemplated, the flooring material manufacturers should be consulted for vapor emission mitigating measures, including but not limited to, concrete mix parameters, vapor retarders and rock layers.

Unless slabs are designed for otherwise, subgrade soil should be presaturated at least five percentage points above optimum moisture content or 130 percent of the optimum moisture content, whichever is greater, to a depth of at least 24-inches. Moisture content must be maintained prior to the placement of concrete slabs.

For preliminary design, slabs should be at least four inches thick, reinforced with No. 3 bars at 18-inches on-center each way. However, these recommendations should be superseded by the design of the structural engineer as per the 1997 UBC. For compliance with UBC, an effective plasticity index (P.I.) of 40 may be utilized for preliminary design.

9.6 Settlement

Total and differential settlement of proposed foundations are expected to not exceed approximately one-inch (1") and one-half of an inch (½") over 100 horizontal feet, respectively, provided that the recommendations presented herein are implemented during grading and final foundation design and construction.

9.7 Corrosion

9.7.1 Metallic

Corrosion testing for metals was not performed at this time, since the final grading conditions of the site will dictate the actual corrosion potential for metals. In general, should ferrous pipe be utilized, the pipes should be encased or wrapped to isolate them from on-site soils. Alternatively, plastic piping may also be used. This should be verified during or after grading by additional laboratory testing.

9.7.2 Concrete

Soluble sulfate testing indicated a result of "negligible" to "moderate" sulfate exposure. Concrete mix design, including but not limited to compressive strength, water cement ratios and cement type, should minimally incorporate the requirements for "moderate" sulfate exposure, as indicated on Table 19-A-4 in the 1997 UBC. This should be verified upon completion of grading by laboratory testing of the exposed subgrade soils. However, for planning purposes (in order to account for soil variability), it is recommended that Type V cement or equivalent be used in structural concrete which comes into contact with the foundation soil.

9.7.3 Sea Breeze

As with developments located near the ocean, salt contained in the air and transmitted in the humidity should be considered as a factor for the design life of this project. A qualified corrosion engineer should provide recommendations to mitigate against the corrosive effects of the ocean on concrete, metal and other materials in the above ground structures.

9.8 Flexible Pavement Design

At this time, it is our understanding that the only pavement to be proposed for the site will be for the proposed Pacific View Drive and related driveways. The on-site parking is to consist primarily of subterranean parking garages. Preliminary pavement design recommendations will be provided in a future report specific to the proposed street improvements.

9.9 Retaining Walls

Retaining walls restraining low expansion soils should be designed to resist an equivalent fluid pressure of 35 pounds per cubic foot for level backfill, and 50 pounds per cubic foot for a 2:1 sloping backfill. Backfill material should consist of granular material (S.E. ≥ 30) and drainage systems should be installed, as shown on the retaining wall details (upper half) presented in Appendix H. For moderate to high-expansive on-site backfill, an equivalent fluid pressure of 55 pounds per cubic foot for level backfill and 70 pounds per cubic foot for 2:1 sloping backfill may be used. On-site backfill soil should be reviewed and approved by the geotechnical consultant prior to utilization. Backfill and drainage systems should be installed as shown on the lower-half of the Retaining Wall Details. For preliminary foundation recommendations, refer to Section 10.3. Surcharge due to vehicular traffic, adjacent structures, and seismic consideration should be added to the above pressures.

When combining frictional and passive resistance to resist lateral loads, one or the other should be reduced by 50 percent.

9.10 Drainage Control

The intent of this section is to provide general information regarding the control of surface water. Based on the moderate to high expansion potential (assumed) of the on-site soils, the regulation of surface water is essential to the satisfactory performance of structures, site improvements, and slopes. The following recommendations are considered minimal.

- Ponding and areas of low flow gradients should be avoided.

- Bare soil within five feet of structures and tops of slopes should have a gradient of at least two percent away from the improvement. For drainage towards the street, a minimum of two percent gradient should be maintained. As an alternative, a gradient of one percent may be used for drainage towards the streets provided that an area drain system designed by the project civil engineer is installed.

- The remainder of the graded areas should be provided with a minimum two percent drainage gradient.

- Positive drainage devices, such as graded swales, paved ditches, and catch basins should be used wherever possible to accumulate and convey water to points of paved discharge areas.

- Concrete walks and flatwork should not obstruct free flow of surface water.

- Brick flatwork should be sealed by mortar or be placed over an impermeable membrane.

- Any planned area drains should be recessed below grade to allow free flow of water into the basin.

- Enclosed, raised or depressed planters or landscape areas should be sealed at the bottom and provided with an ample flow gradient to a drainage device.

- Planters adjacent to a structure should be avoided wherever possible. If planters are to be located adjacent to structures, they should be sealed at the bottom and provided with an ample flow gradient to a drainage device.

- Planting areas at grade should be provided with good positive drainage. Wherever possible, exposed soil areas should be above adjacent paved grades. Drainage devices and curbing should be provided to prevent runoff from adjacent pavement or walks into planted areas.

- Areas with accumulations of sand and/or gravel should have an impermeable bottom seal and be provided with an ample flow gradient to a drainage device.

- Gutter and downspout systems should be provided to capture all discharge from roof areas. The accumulated roof waters should be conveyed to off-site disposal areas by a pipe or concrete swale system.

- Water should be disposed over slopes only if contained in pipes or paved swales discharging to paved disposal areas in benches or at the bottom of slope or other geotechnically acceptable means.

- Site plumbing, landscaping irrigation systems, and sewer lines should be checked and maintained on a continuous basis.

- Water parks, swimming pools and any related drainage systems should be maintained and periodically checked for leaks.

- Landscape watering should be performed judiciously to preclude either soaking or desiccation of soils. The water should be such that it just sustains plant growth without over-watering. Sprinkler systems should be checked periodically to ensure proper working order and should be turned off during the rainy season.

- Surface water should be controlled to the extent that the area beneath the structures always remains dry even during periods of heavy rainfall.

- Adequate drainage gradients, devices and curbing should be provided to prevent runoff from adjacent pavement or walks into planted areas.

- Any rodents found on the site which might impact structural fills or foundation soils should be exterminated and their burrows filled or sealed with soil or slurry.

9.11 Buried Utilities

9.11.1 Trenching

Temporary excavations and trench walls to a depth of four feet may be made vertically without shoring, subject to verification of safety by the contractor. Deeper excavations should be braced, shored or sloped no steeper than 1.5:1 (horizontal to vertical). No surcharge loads should be allowed within five feet from the top of cuts.

All work associated with excavation shoring and bracing should meet the minimum requirements as set forth by CAL-OSHA. Temporary excavation recommendations are provided for general guidelines. Temporary slope and trench excavation construction, maintenance, and safety are the responsibility of the contractor.

Cuts below or near the groundwater surface will need special, condition-specific recommendations. Such recommendations should be formulated once the utility layouts and depths are known.

9.11.2 Trench Bottom Preparation

The bottom of trench excavation should be firm and unyielding and free of deleterious materials. Any disturbed soils should be removed or recompacted to at least 90 percent of the laboratory maximum dry density.

9.11.3 Pipe Bedding

Pipe bedding materials should consist of granular material with sand equivalent of at least 30. In general, open-graded gravel should not be used due to the potential for soil migration into the relatively large void spaces present in this type of material. However, open graded gravel may be used, if desired, provided that filter fabric, such as Mirafi 140NC or equivalent, is used to wrap the gravel.

9.11.4 Trench Backfill

All trench backfill should be placed and compacted to at least 90 percent of the laboratory maximum dry density determined in accordance with ASTM D1557-00. Mechanical compaction is recommended. Ponding or jetting should be avoided, especially in areas supporting structural loads or beneath concrete slabs supported-on-grade, pavements or other improvements.

9.12 Appurtenant Structures

9.12.1 Concrete Flatwork

The following general recommendations may be considered for concrete flatwork, including expansive soils mitigation.

Based upon preliminary expansion index testing and our experience with the site, soils possessing expansion potentials of medium to high may be encountered during grading. Parameters for various expansion potentials are provided.

SIDEWALKS

Expansion Potential	Minimum Concrete Thickness (inches)	Subgrade Pre-Soaking Depth	Reinforcement	Joint Spacing *
EI 0-20 EI 21-50 EI 51-90 EI 91-130	4 (Nominal) 4 (Nominal) 4 (Nominal) 4 (Nominal)	Optimum to 12" 120% of Optimum to 12" 120% of Optimum to 18" 130% of Optimum to 24" (or 5% over optimum, whichever is greater)	N.R. N.R. #3 @ 24" OC, EW** #3 @ 18" OC, EW**	4-5 Feet 4-5 Feet 4-5 Feet 4-5 Feet

* Joints at curves and angle points are recommended

** Optional

N.R. = Not Required

DRIVEWAYS, PATIOS, ENTRYWAYS

Expansion Potential	Minimum Concrete Thickness (inches)	Subgrade Pre-Soaking Depth	Reinforcement	Joint Spacing (Max) *
EI 0-20 EI 21-50 EI 51-90 EI 91-130	4 (Full) 4 (Full) 4 (Full) 4 (Full)	Optimum to 12" 120% of Optimum to 12" 120% of Optimum to 18" 130% of Optimum to 24" (or 5% over optimum, whichever is greater)	#3 @ 36" OC, EW #3 @ 36" OC, EW #3 @ 24" OC, EW #3 @ 18" OC, EW	10 feet 10 feet 10 feet 10 feet

* Joints at curves and angle points are recommended

These recommendations may be superseded by the project architect, structural engineer or the City of Huntington Beach/County of Orange requirements. These recommendations are not intended to mitigate cracking caused by shrinkage and temperature warping.

9.12.2 Landscaping

Summarized below are our optional recommendations for planter area moisture control. The purpose of our recommendations is to reduce the infiltration of irrigation water towards the proposed buildings foundations and slab subgrade.

Each planter or landscape area which is to be situated adjacent to the proposed building foundations should have an area drain system and a moisture barrier installed directly adjacent to the building foundation.

The moisture barrier should consist of a suitable membrane material. The barrier should be installed to a depth of three feet, as measured from the proposed planter area soil surface. The moisture barrier should extend, at a minimum, three feet in both directions past the planter area. The barrier should be permanently attached to the building foundation, utilizing an appropriate waterproof adhesive. The moisture barrier should extend a minimum of 1 to 2 inches above the planter soil surface.

During the installation of the moisture barrier, soil should not be disturbed beyond a 1:1 (horizontal to vertical) projection from the bottom outside edge of the foundation. Soil removed for the purposes of moisture barrier installation should be moisture conditioned as necessary and compacted to 90% relative compaction

10.0 GEOTECHNICAL REVIEW

10.1 Plans and Specifications

This report is intended to be a preliminary study to develop general geotechnical design recommendations for the development as presented on the 50-scale plans, by MVE and dated March 27, 2001. It is our understanding that the proposed development is still in the planning stages and changes in slopes, grades and building locations may change with time. More detailed studies and investigations will need to be performed as the project milestones are reached. It is our understanding that the recommendations in this report are to be used in developing the rough grading plans.

The rough grading and foundation plans should be reviewed and commented on by this office with respect to conformance with the intent of the recommendations presented, and any changes be made to accommodate these geotechnical comments into the final plan set. As already discussed, such reviews may require additional field study and office evaluation of site conditions.

This report and pertinent subsequent studies yet to be performed should be incorporated by reference into rough grading plans. The final rough grading/foundation plans should also be reviewed by this firm.

10.2 Construction Review

The final draft foundation plans for the proposed structures and walls should be reviewed and commented on by this office with respect to conformance with the intent of our recommendations. The final plans should incorporate the results of any such comments.

The geotechnical aspects of the proposed construction should be performed under the observation and testing of this firm. These aspects include pad preparation for foundations, foundation excavation, wall construction and backfilling, utilities, and streets.

11.0 CLOSURE

Our services were performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical engineers and engineering geologists practicing in this or similar localities. No warranty, express or implied, is made as to the conclusions and professional advice included in this report.

The samples taken and used for testing, the observations made, and the in-place field testing performed are believed representative of the project; however, soils and geologic conditions can vary significantly between tested/observed locations.

As in most projects, conditions revealed by excavation may be at a variance with the reported findings. If this occurs, although not anticipated, the changed condition must be evaluated by the project engineering geologist and geotechnical engineer, and the designs affected adjusted as required or alternate designs formulated.

This report is issued with the understanding that it is the responsibility of the owner, or his representative, to ensure that the information and recommendations contained herein are brought to the attention of the project architect, structural engineer and contractor(s), and otherwise incorporated into the plans and specifications. Similarly, it is also the responsibility of the owner or his representative to take the necessary steps to ensure these recommendations are carried out during construction.

This firm does not practice or consult in the field of safety engineering or surveying. We do not direct the contractors operations, and thus, we cannot be responsible for actions of other than our staff on the site.

Geotechnical services are provided by ZKCI in accordance with generally accepted professional engineering and geologic practice in the area where these services are to be rendered. Client acknowledges that the present standard in the engineering and geologic and environmental profession does not include a guarantee of perfection and, except as expressly set forth in the Conditions above, no warranty, expressed or implied, is extended by ZKCI.

All excavations used for subsurface exploration were backfilled prior to leaving the site. As with any backfill, consolidation and subsidence may result in depression of the excavation area and a potentially hazardous condition. The client and/or owner of the property are hereby advised to periodically examine the excavation areas, and if necessary backfill any resulting depressions. ZKCI shall not be responsible for injury or damage resulting from subsidence of backfill.

Geotechnical reports are based on the project description and proposed scope of work as described in the proposal. Our conclusions and recommendations are based on the results of the field, laboratory, and office studies, combined with an interpolation and extrapolation of soil conditions as described in the report. The results reflect our geotechnical interpretation of the limited direct evidence obtained. Our conclusions and recommendations are made contingent upon the opportunity for ZKCI to continue to provide geotechnical services beyond the scope in the proposal to include all geotechnical services. If parties other than ZKCI are engaged to provide such services, they must be notified that they will be required to assume complete responsibility for the geotechnical work of the project by concurring with the recommendations in our report or by providing alternate recommendations.

All locations of borings/exploratory trenches, cut/fill transitions, limits of fill, verification of overexcavations, contacts, elevations, etc., are represented herein to the best of our abilities. The approximate locations depicted on all plates and figures are based upon available control as provided in the field by others. Where no information was provided by others, locations were approximated using limited measuring methods and crude instrumentation. We do not verify the locations or elevations reported herein as accurate in survey or void of error. ZKCI assumes no responsibility for any future costs associated with errors in the area of survey.

It is the readers responsibility to verify the correct interpretation and intention of the recommendations presented herein. ZKCI assumes no responsibility for misunderstandings or improper interpretations that result in unsatisfactory or unsafe work products. It is the readers further responsibility to acquire copies of any supplemental reports, addenda or responses to public agency reviews that may supersede recommendations in this report.

ZEISER
KLING
Consultants, Inc.

March 4, 2002 PN 01039-00

Mr. Ethen Thatcher
Makar Properties, LLC
4100 MacArthur Boulevard, Suite 150
Newport Beach, California 92660

SUBJECT: Addendum report and response to city comments regarding "Preliminary Geotechnical Investigation, Proposed Pacific City, Northeastern Corner of 1st Street and Pacific Coast Highway, City of Huntington Beach, California," prepared by Zeiser Kling Consultants, Inc., dated November 19, 2001.

Dear Mr. Thatcher:

In accordance with your request and authorization, Zeiser Kling Consultants, Inc. (ZKCI) has reviewed the comments presented in the letter prepared by The City of Huntington Beach, Department of Planing, dated February 26, 2002. This review was performed on our report titled, "Preliminary Geotechnical Investigation, Proposed Pacific City, Northeastern Corner of 1st Street and Pacific Coast Highway, City of Huntington Beach, California," prepared by Zeiser Kling Consultants, Inc., Project Number 01039-00, dated November 19, 2001. Our responses to these comments are as follows:

Comment 1:
Dewatering and mitigation requirements for the disposal of groundwater during grading and construction operations are only briefly mentioned. In view of the operations already ongoing on the adjacent properties to the east, additional recommendation for dewatering and disposal of the water can be provided.

Response:
Our report was intended to be a "preliminary" level report to identify the issues and to give general geotechnical recommendations for planning purposes. More site-specific studies for the proposed hotel, commercial area and residential area will be performed once more final plans are provided. It is also our intention to perform a more detailed study for dewatering and groundwater issues in the near future, if future plans indicate that it is required.

The current plans used for this report show the proposed bottom of the parking garages to be at or above the current groundwater level in the southwestern portion of the site. As mentioned in Section 8.3.4 of our report, this is the only area that may require dewatering, and will require

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Geotechnical Engineering • Engineering Geology • Materials Testing and Inspection

additional study. For the purpose of addressing the City's concerns, we are providing the following as a "potential" recommendation.

If dewatering is required, it is our opinion that the water can be stored and treated on-site, if any treatment is required. This water can be used for the construction on site, and any extra water, once acceptably treated, may be discharged into the storm drain systems. If this is unacceptable, it may also possibly be reinjected into the ground. See our response to Comment 2 below.

Dewatering and water storage/treatment/handling will be a focus of our upcoming detailed studies. It is our intent and approach to minimize disturbance or pumping of the groundwater at the site.

Comment 2:
The subject of contaminated water is briefly addressed but assumes the issues have been addressed by other consultants. We suggest that reference should be made to the specific documentation supporting that assumption and the results found in those studies and the remediation work performed.

Response:
Based on our review of the AGRA Earth and Environmental, Inc. report titled "Letter Report For Shallow Groundwater Monitoring Well Installation and Testing, Atlanta/Huntington Development Project, Huntington Beach, California," Job Number 0-214-801200, dated May 12, 2000, it is our opinion that the groundwater on site is treatable on-site for discharge.

They tested groundwater samples from three wells, and found that these samples contained trace oil (1 to 2 parts-per-million [ppm]). Well 1 had indicated elevated copper levels, and Well 2 was highly saline, but these levels, when mixed with other "clean" groundwater from the dewatering operations and/or water from other sources, maybe suitable diluted for discharge, once treated.

We concur with AGRA's opinion that this water can be treated on site using settling tanks and carbon finishing) for General National Pollution Discharge Elimination System (NPDES) discharge. We also concur with their findings, conclusions and recommendations with respect to groundwater issues and quality.

We thank the City of Huntington Beach and their reviewers for their considerations and thoughtful review comments. It is hoped that the responses presented herein and to be provided in our focused studies will be sufficient to address the City's concerns.

We look forward to beginning our focused studies with respect to geotechnical and geohydrologic conditions at the site.

The opportunity to be of continued service to Makar Properties, LLC is appreciated. Please contact the undersigned with questions or comments.

Respectfully submitted,

ZEISER KLING CONSULTANTS, INC.

Michael W. Laney
Senior Project Engineer
G.E. 2539
Expires 6/30/05

Larry E. Fanning
Principal Engineering Geologist
R.G. 6118; C.E.G. 1907
R.E.A. 04677
Expires 1/31/03 and 6/30/02

MWL:LEF:wo

Attachments: A- City of Huntington Beach Comments Letter

Distribution: (6) Addressee

Attachment A

City of Huntington Beach
Comments Letter

City of Huntington Beach

2000 MAIN STREET CALIFORNIA 92648

DEPARTMENT OF PLANNING

Phone 536-5271
Fax 374-1540
374-1648

February 14, 2002

Ethen Thatcher
Makar Properties, LLC
4100 MacArthur Blvd., Ste. 200
Newport Beach, CA 92660

Subject: Preliminary Geotechnical Investigation

Dear Ethen:

The City has completed peer review of the Preliminary Geotechnical Investigation received January 8th. Please revise the report, or provide an addendum, to address the comments listed below.

1. Dewatering and the mitigation requirements for the disposal of groundwater during grading and construction operations are only briefly mentioned. In view of the operations already ongoing on the adjacent properties to the east, additional recommendations for dewatering and the disposal of the water can be provided.

2. The subject of contaminated water is briefly addressed but assumes the issues have been addressed by other consultants. We suggest that reference should be made to the specific documentation supporting that assumption and the results found in those studies and the remediation work performed.

The Public Works Department can assist you should you have any questions regarding these comments. Thank you.

Sincerely,

Mary Beth Broeren
Principal Planner

Cc: Scott Hess, Planning Manager

Appendices to this report are available for review at the City of Huntington Beach and City of Huntington Beach Central and Main libraries.