APPENDIX G

GEOTECHNICAL AND HYDROGEOLOGICAL REPORTS

This appendix includes the geotechnical and hydrogeological reports that were prepared by the Applicant's consultants Byer Geotechnical, Inc. and Thomas Harder & Co. These technical documents informed the analysis provided in Section 3.5, *Geology and Soils* and Section 3.7, *Hydrology and Water Quality*.

G1: Geotechnical Engineering Exploration (Byer Geotechnical, Inc. 2015)

G2: Hydrogeologic Evaluation in Support of Environmental Documentation for the Strand and Pier Hotel (Thomas Harder & Co. 2016)

BYER GEOTECHNICAL, INC.

September 23, 2015
BG 21877

Bolour Associates
8383 Wilshire Boulevard, Suite 920
Beverly Hills, California 90211

Attention: Mr. Michael Mathews, Acquisitions Associate

Subject

Transmittal of Geotechnical Engineering Exploration
Proposed Three-Story Hotel Building over Two Basement Levels
Lots 1 - 9, 19, 20, and the Westerly 30 Feet of Lots 30 and 31, Block 13, Hermosa Beach Tract
1250 - 1272 The Strand, 11 and 19 Pier Avenue, and 20 - 32 13th Street
Hermosa Beach, California

Dear Mr. Mathews:

Byer Geotechnical has prepared our report dated September 23, 2015, which describes the geotechnical engineering conditions with respect to the proposed project. This report is intended to replace the February 10, 2014, report. All copies of the February 10, 2014, report should be discarded. The reviewing agency for this document is the City of Hermosa Beach, Building and Safety Division. Four copies of the report are enclosed.

It is our understanding that Bolour Associates will file the report with the City of Hermosa Beach. Please review the report carefully prior to submittal to the governmental agency. Questions concerning the report should be directed to the undersigned. Byer Geotechnical appreciates the opportunity to offer our consultation and advice on this project.

Very truly yours,
BYER GEOTECHNICAL, INC.

Raffi S. Babayan
Senior Project Engineer

BYER GEOTECHNICAL, INC.

GEOTECHNICAL ENGINEERING EXPLORATION

PROPOSED THREE-STORY HOTEL BUILDING OVER TWO BASEMENT LEVELS

LOTS 1 - 9, 19, 20, AND THE WESTERLY 30 FEET OF LOTS 30 AND 31, BLOCK 13

HERMOSA BEACH TRACT

1250 - 1272 THE STRAND, 11 AND 19 PIER AVENUE, AND 20 - 32 13TH STREET

HERMOSA BEACH, CALIFORNIA

FOR BOLOUR ASSOCIATES

BYER GEOTECHNICAL, INC., PROJECT NUMBER BG 21877

SEPTEMBER 23, 2015

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INTRODUCTION

This report has been prepared per our signed Agreement and summarizes findings of Byer Geotechnical, Inc., geotechnical engineering exploration performed on the subject site. The purpose of this study is to evaluate the nature, distribution, and engineering properties of the earth materials underlying the site with respect to construction of a three-story hotel building over two basement levels. This report is intended to assist in the design and completion of the proposed project and to reduce geotechnical risks that may affect the project. The professional opinions and advice presented in this report are based upon commonly accepted exploration standards and are subject to the AGREEMENT with TERMS AND CONDITIONS, and the GENERAL CONDITIONS AND NOTICE section of this report. No warranty is expressed or implied by the issuing of this report.

PROPOSED PROJECT

The scope of the proposed project was determined from consultation with Mr. Michael Mathews of Bolour Associates and the preliminary plans prepared by HKS Architects, dated March 20, 2015. Final plans have not been prepared and await the conclusions and recommendations of this report. The project consists of construction of a three-story hotel building over two basement levels. The basement is planned for parking and event rooms. The footprint of the basement is planned to occupy the entire site, as shown on the enclosed Site Plan. Retaining walls from 18 to 28 feet high are planned to support the excavation for the basement. Columns loads (dead and live) on foundations are expected to be moderate. The existing one- and two-story restaurant and retail stores and associated at-grade parking areas, and one-story buildings adjacent to 13th Street are to be removed. In addition, a portion of Beach Drive and the west end of 13th Court, which both traverse the subject site, are planned to be abandoned.

EXPLORATION

The scope of the field exploration was determined from our initial site visit and consultation with Mr. Michael Mathews of Bolour Associates. The preliminary plans prepared by HKS Architects, dated March 20, 2015, were a guide to our work on this project. Exploration was conducted using techniques normally applied to this type of project in this setting. This report is limited to the area of the exploration and the proposed project, as shown on the enclosed Site Plan and cross sections. The scope of this exploration did not include an assessment of general site environmental conditions for the presence of contaminants in the earth materials and groundwater. Conditions affecting portions of the property outside the area explored are beyond the scope of this report.

Exploration was conducted on January 2, 2014, with the aid of an electronic piezocone penetrometer (CPT) and a hollow-stem-auger drill rig. It included drilling three borings and advancing three CPT soundings to approximate depths of 35 to 50½ feet below existing grade. Samples of the earth materials were obtained and delivered to our soils engineering laboratory for testing and analysis.

The boring tailings were visually logged by the project soils engineer. Following drilling, logging, and sampling, the borings were backfilled and mechanically tamped, and patched with asphalt and concrete, where applicable. The CPT soundings were backfilled with bentonite chips and patched with asphalt.

Office tasks included laboratory testing of selected soil samples, review of published maps and photos for the area, review of our files, preparation of cross sections, preparation of the Site Plan, engineering analysis, and preparation of this report. Earth materials exposed in the borings are described on the enclosed Log of Borings. Appendix I contains a discussion of the laboratory testing procedures and results. Appendix II contains the results of the liquefaction analysis and the cone penetrometer results and interpretations.

The proposed project and the locations of the borings and CPT soundings are shown on the enclosed Site Plan. Subsurface distribution of the earth materials and the proposed project are shown on Sections A and B.

SITE DESCRIPTION

The subject site consists of a trapezoidal-shaped and relatively-level parcel located just east of Hermosa Beach and the Pacific Ocean in the city of Hermosa Beach, Los Angeles County, California (33.8622° N Latitude, 118.4016° W Longitude). The site consists of thirteen lots and portions of Beach Drive and 13th Court. As depicted on the enclosed Aerial Vicinity Map, the site is bounded by 13th Street on the north, Pier Avenue on the south, an existing two-story commercial building and public parking lot on the east, and The Strand on the west. Elevation across the site varies from about 13 to 15 feet above sea level. The site is located approximately 4.8 miles south of the Glenn Anderson (105) Freeway and 4.7 miles west of the San Diego (405) Freeway. Several one- and two-story restaurant and retail stores occupy the majority of the site, and an at-grade parking lot occupies the southwest corner of the site. A portion of Beach Drive and the west end of 13th Court traverse the site from north to south and from east to west, respectively, as shown on the enclosed Aerial

Vicinity Map. The surrounding area has been developed with one- and two-story commercial establishments on both sides of Pier Avenue, as well as single- and multi-family residential buildings to the east.

Past grading on the site has consisted of placing minor amounts of fill to prepare a level pad for the existing structures. The site is devoid of vegetation, except a raised planter at the southwest corner containing a small tree. Surface drainage is by sheetflow runoff down the contours of the land to the west.

GROUNDWATER

Groundwater was encountered in the borings and CPT soundings at approximate depths of 8 to 11 feet below existing grade. Pore pressure dissipation tests in CPT 1 to 3 indicate that the groundwater is between 8.7 and 10.8 feet below grade. In *Seismic Hazard Zone Report 031,* the California Geological Survey (CGS) has estimated the historically-highest groundwater level at the site was 10 feet, or less, below ground surface (CGS, 1998). Fluctuations in groundwater levels occur due to the tide, as well as variations in climate, irrigation, development, and other factors not evident at the time of the exploration. Groundwater levels may also differ across the site. Groundwater can saturate earth materials causing subsidence or instability of slopes.

EARTH MATERIALS

Fill

Fill was not encountered in the borings drilled on the subject site, but should be expected locally as utility trench backfills.

Beach Sand

Natural beach sand deposits, common for the coastline (Poland, 1959), underlie the subject site. The beach sand consists of poorly- to well-graded sand that is light to dark greenish-brown and gray-brown, slightly moist, becoming saturated below groundwater. The beach sand is characterized as medium dense to very dense (see enclosed Log of Borings).

GENERAL SEISMIC CONSIDERATIONS

Regional Faulting

The subject site is located in an active seismic region. Moderate to strong earthquakes can occur on numerous local faults. The United States Geological Survey, California Geological Survey (CGS), private consultants, and universities have been studying earthquakes in southern California for several decades. Early studies were directed toward earthquake prediction and estimation of the effects of strong ground shaking. Studies indicate that earthquake prediction is not practical and not sufficiently accurate to benefit the general public. Governmental agencies now require earthquake-resistant structures. The purpose of the code seismic-design parameters is to prevent collapse during strong ground shaking. Cosmetic damage should be expected.

Southern California faults are classified as "active" or "potentially active." Faults from past geologic periods of mountain building that do not display evidence of recent offset are considered "potentially active." Faults that have historically produced earthquakes or show evidence of movement within the past 11,000 years are known as "active faults." No known active faults cross the subject site, and the site is not located within a currently-designated Alquist-Priolo Earthquake Fault Zone (CGS, 2000). Therefore, the potential for future surface rupture onsite is expected to be very low.

The known regional local active faults that could produce the most significant ground shaking on the site include the Palos Verdes, Newport-Inglewood, Santa Monica, Malibu Coast, and Anacapa-

Dume Faults. Forty-eight faults were found within a 100-kilometer-radius search area from the site using EZ-FRISK V7.62 computer program. The results of seismic-source analysis are listed in Appendix II. The closest mapped "active" fault that could affect the site is the Palos Verdes Fault, a Type B fault that is located 2.7 kilometers west of the site. The Palos Verdes Fault is capable of producing a maximum moment magnitude of 7.3 and an average slip rate of 3.0 ± 1.0 millimeters per year (Cao et al., 2003). The San Andreas Fault, a Type A fault, is located 80.5 kilometers northeast of the site. General locations of regional active faults with respect to the subject site are shown on the enclosed Regional Fault Map.

Seismic Design Coefficients

The following table lists the applicable seismic coefficients for the project based on the California Building Code:

SEISMIC COEFFICIENTS (2013 California Building Code - Based on ASCE Standard 7-10)		
Latitude = 33.8622° N Longitude = 118.4016° W	Short Period (0.2s)	One-Second Period
Earth Materials and Site Class from Table 20.3-1, ASCE Standard 7-10	Beach Sand - D	
Mapped Spectral Accelerations from Figures 1613.3.1 (1) and 1613.3.1 (2) and USGS	S_s = 1.637 (g)	S_1 = 0.625 (g)
Site Coefficients from Tables 1613.3.3 (1) and 1613.3.3 (2) and USGS	F_A = 1.0	F_V = 1.5
Maximum Considered Spectral Response Accelerations from Equations 16-37 and 16-38, 2013 CBC	S_{MS} = 1.637 (g)	S_{M1} = 0.938 (g)
Design Spectral Response Accelerations from Equations 16-39 and 16-40, 2013 CBC	S_{DS} = 1.091 (g)	S_{D1} = 0.625 (g)
Maximum Considered Earthquake Geometric Mean (MCE_G) Peak Ground Acceleration, adjusted for Site Class effects	PGA_M = 0.638 (g)	

Reference: U.S. Geological Survey, **Geologic Hazards Science Center, U. S. Seismic Design Maps,** http://earthquake.usgs.gov/designmaps/us/application.php

The mapped spectral response acceleration parameter for the site for a 1-second period (S_1) is less than 0.75g. The design spectral response acceleration parameters for the site for a 1-second period (S_{D1}) is greater than 0.20g, and/or the short period (S_{DS}) is greater than 0.50g. Therefore, the project is considered to be in Seismic Design Category D.

The principal seismic hazard to the proposed project is strong ground shaking from earthquakes produced by local faults. Modern buildings are designed to resist ground shaking through the use of shear panels, moment frames, and reinforcement. Additional precautions may be taken, including strapping water heaters and securing furniture to walls and floors. It is likely that the subject property will be shaken by future earthquakes produced in southern California.

Ground Motion

Probabilistic seismic hazard deaggregation analysis was performed on the subject site. Seismic parameters were determined using currently available earthquake and fault information utilizing data from the United States Geological Survey (USGS) National Seismic Hazard Mapping Project (USGS, 2008). An averaging of three Next Generation Attenuation relations (Chiou-Youngs, 2008; Boore-Atkinson, 2008; and Campbell-Bozorgnia, 2008) was incorporated in the analysis. An average shear-wave velocity (Vs30) of 330 meters-per-second (Site Class D) was used in the analysis. Hazard deaggregation indicates a predominant mean earthquake magnitude of 6.68 (Mw) at a distance of 17.1 kilometers. The Peak Horizontal Ground Acceleration (PHGA) with a 10-percent probability of exceedance in 50 years is estimated to be 0.35g on the subject site. These ground motions could occur at the site during the life of the project. Results of the analysis are graphically presented in the enclosed "Seismic Hazard Deaggregation Chart," along with a summary of the calculations (Appendix II).

Based on a Site Class D, the MCE_G peak ground acceleration adjusted for Site Class effects, PGA_M, is 0.638g. The horizontal pseudo-static seismic coefficient (k_h) was taken as one-third of the PGA_M (0.21g) and was used in the seismic calculations for the cantilever and basement retaining walls.

Liquefaction

The CGS has mapped the site within an area where historic occurrence of liquefaction or geological, geotechnical, and groundwater conditions indicate a potential for permanent ground displacement such that mitigation as defined in Public Resources Code Section 2693 (c) would be required (CGS, 1999).

Liquefaction is a process that occurs when saturated sediments are subjected to repeated strain reversals during an earthquake. The strain reversals cause increased pore water pressure such that the internal pore pressure approaches the overburden stress and the shear strength approaches zero. Liquefied soils may be subject to flow or excessive strain, which may induce settlement. Liquefaction occurs in soils below the groundwater table. Soils commonly subject to liquefaction include loose to medium-dense sand and silty sand. Predominantly fine-grained soils, such as silts and clay, are less susceptible to liquefaction. Generally, soils with fines content (percent passing the No. 200 sieve) greater than 35 percent are not considered susceptible to liquefaction. In addition, soils with Plasticity Index (PI) greater than or equal to 12 and moisture content less than 85 percent of the Liquid Limit (LL) are not considered susceptible to liquefaction (CGS, 2008). Soils data collected in Boring 1 were utilized to quantify the liquefaction potential of the site. A ground acceleration of 0.35g and a design magnitude earthquake of 7.19 (M_w) were used for the analyses. It was assumed that groundwater remains at the current average groundwater level, nine feet below the ground surface.

A liquefaction potential analysis based upon SPT data from Boring 1 is presented in Appendix II on the enclosed plate entitled "Liquefaction Susceptibility Analysis: SPT Method." The column labeled "Factor of Safety" lists the calculated safety factor of each 2½-foot-thick soil layer encountered in Boring 1. The stresses and safety factor for liquefaction were calculated using the methodology of Idriss and Boulanger (2008) and Special Publication 117A (CGS, 2008). A factor of safety of 11.3 against liquefaction was used to differentiate between potentially-liquefiable and non-liquefiable soil layers.

Because of the limitations of SPT sampling frequency, the potential inconsistencies in performing the SPT procedure, and to more precisely identify and define soil layers with the potential to be susceptible to liquefaction, a liquefaction analysis was also performed using cone penetrometer (CPT) data and is included in Appendix II on the enclosed plate entitled "Liquefaction Susceptibility Analysis: CPT Method." The column "Factor of Safety" of the analysis lists the calculated safety factor of each one-foot-thick layer of soil encountered in the CPT soundings. The stresses and safety factor for liquefaction were calculated using the methodology of P. K. Robertson (2006), and Special Publication 117A (CGS, 2008). The liquefaction safety factors for each soil layer below the subgrade elevation are presented in the "Liquefaction Susceptibility Analysis - CPT Method." A factor of safety of 1.3 against liquefaction was used to differentiate between potentially-liquefiable and non-liquefiable soil layers.

The liquefaction analysis based upon the SPT data from Boring 1 indicates that there are 2½-foot-thick layers at various depths that may be susceptible to liquefaction. The CPT data indicates that several one-foot-thick layers of soil at various depths below existing grade may be susceptible to liquefaction. The results are shown in the following table.

Depths and Thicknesses of Soil Layers Potentially Susceptible to Liquefaction (Based on SPT and CPT Data)		
Boring/CPT #	Thickness (feet)	Depths Below Grade (feet)
B1	2.5	17½, 20, 22½, 25, 27½, 32½
CPT1	1.0	21, 26, 27, 28, 29, 30, 31, 36, 37, 38, 39
CPT2	1.0	21, 25, 26, 27, 28, 29, 30, 31, 32
CPT3	1.0	12, 17, 18, 19, 24, 25, 26, 29, 32, 33, 34

Dynamic Settlement

Earthquake-induced volumetric strain and dissipation of pore pressure in saturated silts and sands after liquefaction can result in settlement. The potential for liquefaction-induced settlement was calculated using the methodology of Tokimatsu and Seed (1987) for the SPT data, and Zhang et al. (2002) for the CPT data. The seismic settlement potentials were calculated for all granular soil layers at depths below the groundwater table and with a factor of safety for liquefaction less than 1.3. Depending on the final depth of the basement levels, the total and differential dynamic settlements are determined based on the potentially-liquefiable soil layers that will remain below the basement levels. Accordingly, the total and differential dynamic settlements are shown in the following table:

Calculated Total and Differential Dynamic Settlements		
Depth of Basement (feet)	Total Dynamic Settlement (inches)	Differential Dynamic Settlement (inches)
18	1.80 (CPT3) to 3.20 (B1)	1.4
28	0.58 (CPT2) to 1.45 (CPT1)	0.87

The maximum differential dynamic settlement potential is taken as the difference between the minimum and maximum total dynamic settlements.

Lateral Spreading Hazard

Liquefied soils may be subject to lateral spreading flow failure where adjacent to slopes or "free-faces" such as steep slopes or embankments. The subject site is not located adjacent to free-faces, slopes, and canals, and a lateral spreading flow failure is not indicated for the potentially-liquefiable alluvial soils. Therefore, it is the opinion of Byer Geotechnical, Inc., that the lateral spreading hazard at the site is considered very low and no mitigation as defined in Public Resources Code Section 2693(c) is required for lateral spreading.

Tsunamis

Tsunamis are waves generated in large bodies of water by fault displacement or major ground movement. Tsunamis are historically rare events in southern California. Based on the Tsunami

Inundation Map published by the California Emergency Management Agency (Cal EMA), the west portion of the site is located within a Tsunami Inundation Area (Cal EMA, 2009).

CONCLUSIONS AND RECOMMENDATIONS

General Findings

The conclusions and recommendations of this exploration are based upon review of the preliminary plans, review of published maps, three borings, three CPT soundings, research of available records, laboratory testing, engineering analysis, and years of experience performing similar studies on similar sites. It is the finding of Byer Geotechnical, Inc., that development of the proposed project is feasible from a geotechnical engineering standpoint, provided the advice and recommendations contained in this report are included in the plans and are implemented during construction.

The liquefaction analysis indicates a potential for liquefaction that could result in up to 3.2 inches of dynamic settlement in the event of a strong local earthquake. A mat foundation may be used to support the proposed three-story hotel building over two basement levels and mitigate the 3.2 inches of seismic settlement. Depending on the actual elevations of the basement levels, the bottom of the mat foundation should be deepened so as to bear below the potentially-liquefiable soil layers identified at elevations -8.0 and -16.0 feet, as shown on the enclosed Sections A and B.

An alternative foundation system that would reduce or eliminate the potential for liquefaction-induced dynamic settlement could consist of driven precast concrete piles. Pile tips should extend below elevation of -25 feet.

Soils to be exposed at the finished grade of the lowest basement level are expected to exhibit a very low expansion potential.

Geotechnical issues affecting the project include temporary excavations ranging from 20½ to 30½ feet, including an estimate of the thickness of the mat foundation or grade beams/pile caps. Temporary shoring is required to facilitate the construction of the basement. Sheet-pile walls have been successful where the retaining soil consists of sand that extends below groundwater level. Recommendations for temporary shoring are included in the "Temporary Excavations" section of this report.

Groundwater will be encountered in the bottom of the proposed basement. A temporary dewatering system is required during construction. The groundwater level should be maintained at least 10 feet below the lowest basement level to facilitate the construction of the foundation system. The dewatering system should be designed by a hydrogeologist.

FOUNDATION DESIGN

Mat Foundation

A mat foundation may be used to support the proposed building, provided it is founded in undisturbed natural soil. The minimum thickness of the mat should be 30 inches. The structural engineer may require a greater thickness. The following chart contains the recommended design parameters.

Bearing Material	Minimum Thickness of Mat Foundation (Inches)	Allowable Vertical Bearing (psf)	Coefficient of Friction	Passive Earth Pressure (pcf)	Maximum Earth Pressure (psf)
Beach Sand	30	2,000	0.40	250	2,000

For bearing calculations, the weight of the concrete may be neglected. A hydrostatic pressure applied from elevation six feet to the bottom of the mat foundation should be incorporated into the design of the mat foundation based on the current basement grade planned. The final thickness of the mat foundation may be on the order of four to eight feet.

The bearing value shown above is for the total of dead and frequently applied live loads and may be increased by one-third for short duration loading, which includes the effects of wind or seismic forces. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

The bottom of the mat foundation excavation should be free from loose material and construction debris, and should be approved by the geotechnical engineer prior to placing forms, steel, or concrete.

Modulus of Subgrade Reaction

The allowable modulus of subgrade reaction, k_1, is 250 kips-per-cubic-foot for a 12-inch by 12-inch footing. The modulus should be reduced for larger footings, such as the proposed mat. For rectangular footings of dimensions B x L, the following formula may be used (Bowles, 1997):

$$k_s = k_1 * (m + 0.5) / (1.5 * m)$$

where k_s = Modulus of subgrade reaction for a full-size mat foundation,

m = L / B.

Foundation Settlement

Settlement of the mat foundation system is expected to occur on initial application of loading. An estimated static settlement of one-quarter to one-half of an inch may be anticipated. Differential static settlement should not exceed one-quarter of an inch across the building footprint.

Based on the liquefaction analysis performed on the site, the combined total and differential settlements (static and dynamic) estimated on the subject site are listed in the following table:

Estimated Combined Total and Differential Settlements		
Basement Depth Below Grade (feet)	Total Settlement (inches)	Differential Settlement (inches)
18	2.05 to 3.70	1.65
28	0.83 to 1.95	1.12

Deepened Foundations - Driven Piles

As an alternative to a mat foundation, to mitigate potentially excessive dynamic settlement, driven precast concrete piles may be used to support the proposed building. Driven piles should be a minimum of 12 inches square and should be embedded into the alluvium a minimum of five feet below the lowest potentially-liquefiable soil layer encountered at an approximate elevation of -24 feet.

Pile tip capacity, unit skin friction, and total skin capacity analyses were checked for 12-, 14-, and 16-inch-square piles utilizing the cone penetrometer test (CPT) data. Six CPT-based pile-design methodologies were averaged in the analysis (Aoki and De Alencar, 1975; Clisby et. al. (1978); De Ruiter and Beringen, 1979; Prince and Wardle, 1982; Schmertmann, 1978; and Tumay and Fakhroo, 1982). A list of references for these methodologies is enclosed.

Results of the pile capacity analysis are illustrated in the enclosed Pile Axial Capacity Charts 1 and 2 for top-of-pile depths of 18 and 28 feet, respectively (see Calculation Sheets #1 and #2, Appendix III). The structural engineer may design piles that are deeper or larger in cross section depending on actual design loads.

Downdrag

Piles installed through potentially-liquefiable soil layers may be subject to downdrag forces (negative skin friction) along the shaft, caused by the downward movement of the soil above potentially-liquefiable soil layers during strong ground shaking. Potentially-liquefiable layers were encountered onsite at various depths between 17 and 39 feet below existing grade, as shown on the table in the "Liquefaction" section of this report. The downdrag force may be estimated by assuming a negative skin friction over the surface area of the pile for that portion above the lowest potentially-liquefiable soil layers. As a result, the downdrag force should be applied to that portion of the pile from the pile cap at the top down to elevation -24 feet at the bottom of the lowest liquefiable soil layer. A negative skin friction of one-half the downward skin friction shown in Calculation Sheets #1 and #2 may be used to determine the downdrag force on the piles.

Lateral Design

The friction value is for the total of dead and frequently applied live loads and may be increased by one-third for short duration loading, which includes the effects of wind or seismic forces. Resistance to lateral loading may be provided by passive earth pressure within natural soil below the excavation.

Passive earth pressure may be computed as an equivalent fluid having a density of 250 pounds-per-cubic-foot. The maximum allowable earth pressure is 4,000 pounds-per-square-foot. For design of isolated piles, the allowable passive and maximum earth pressures may be increased by 100 percent. Piles spaced more than 2½-pile diameters on center may be considered isolated.

Foundation Settlement - Driven Piles

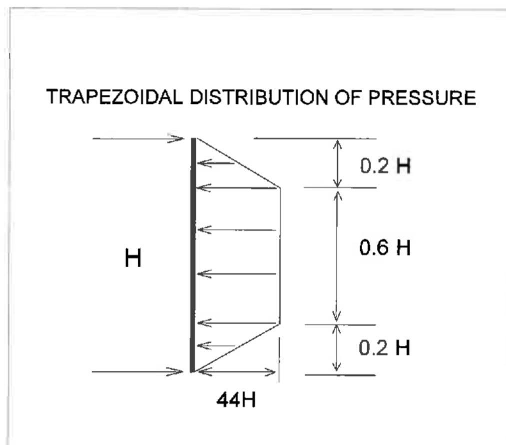
Settlement of the driven pile foundation system is expected to occur on initial application of loading. A total settlement of one-quarter to one-half of an inch may be anticipated. Differential settlement should not exceed one-quarter of an inch. Since the piles are to be embedded below the lowest potentially-liquefiable soil layer, dynamic settlement need not be considered.

RETAINING WALLS

General Design

Cantilever retaining walls up to 16½ feet high, with a level backslope and no surcharge, may be designed for an active equivalent fluid pressure of 61 pounds-per-cubic-foot (see Calculation Sheet #3, Appendix III). A hydrostatic pressure applied from elevation six feet to the bottom of the retaining wall should be incorporated into the design of the retaining walls.

Proposed basement walls, which will be restrained, should be designed for an at-rest lateral earth pressure of 44H, where H is the height of the wall. The diagram illustrates the trapezoidal distribution of earth pressure. A hydrostatic pressure applied from elevation six feet to the bottom of the wall should be incorporated into the design of the basement walls.



Seismic analysis of cantilever and restrained retaining walls indicates that no additional loading due to seismic forces is required, since the calculated seismic thrust is less than the static active and at-rest design thrusts for retained heights of 16½ to 28 feet (see Calculation Sheets #4, #5 and #6, Appendix III).

Backfill

Retaining wall backfill should be compacted to a minimum of 90 percent of the maximum dry density, as determined by ASTM D 1557-12, or equivalent. Where access between the retaining wall and the temporary excavation prevents the use of compaction equipment, retaining walls should be backfilled with ¾-inch crushed gravel to within two feet of the ground surface. Where the area between the wall and the excavation exceeds 18 inches, the gravel must be vibrated or wheel-rolled, and tested for compaction. The upper two feet of backfill above the gravel should consist of a compacted-fill blanket to the surface. Restrained walls should not be backfilled until the restraining system is in place.

Foundation Design

Retaining walls should be supported on either the mat foundation or driven piles, per the recommendations included in the "Mat Foundation" and "Deepened Foundations" sections of this report.

Retaining Wall Deflection

It should be noted that non-restrained retaining walls can deflect up to one percent of their height in response to loading. This deflection is normal and results in lateral movement and settlement of the backfill toward the wall. The zone of influence is within a 1:1 plane from the bottom of the wall. Hard surfaces or footings placed on the retaining wall backfill should be designed to avoid the effects of differential settlement from this movement. Decking that caps a retaining wall should be provided with a flexible joint to allow for the normal deflection of the retaining wall. Decking that does not cap a retaining wall should not be tied to the wall. The space between the wall and the deck will require periodic caulking to prevent moisture intrusion into the retaining wall backfill.

TEMPORARY EXCAVATIONS

Temporary excavations will be required during grading to construct the basement of the proposed building. The excavations will range from 20½ to 30½ feet in height, including an estimate of the mat foundation thickness, and will expose beach sand. Vertical excavations in the beach sand are not recommended. Vertical excavations will require the use of temporary shoring. The use of driven sheet piles is recommended. Design values can be found in the "Sheet-Pile Walls" design section below.

The geotechnical engineer should be present during grading to see temporary slopes. Water should not be allowed to pond on top of the excavations nor to flow toward them. No vehicular surcharge should be allowed within three feet of the top of the cut.

Sheet-Pile Walls

Driven, steel sheet-pile walls may be used as temporary shoring to facilitate the construction of the basement level of the proposed building. Lateral earth pressure analysis was performed to determine the active and passive pressures on the sheet-pile walls. Groundwater was considered in the analysis. A factor of safety of 1.25 was used for design of temporary shoring. Horizontal force and moment equilibrium analysis was also performed for the stability of the sheet-pile walls. The results of the analysis indicate a minimum depth of wall penetration of 36.1 and 50.8 feet below the excavation for retained heights of 20½ and 30½ feet, respectively. The results are shown on the enclosed Sheet Pile Wall Pressure Distribution Charts (Calculation Sheets #7 and #8, Appendix III).

FLOOR SLABS

Floor slabs should be cast over a firm compacted subgrade and reinforced with a minimum of #4 bars on 16-inch centers, each way. Slabs that will be provided with a floor covering should be protected by a polyethylene plastic vapor barrier. A low-slump concrete may be used to minimize possible curling of the slab. The concrete should be allowed to cure properly before placing vinyl or other moisture-sensitive floor covering.

It should be noted that cracking of concrete slabs is common. The cracking occurs because concrete shrinks as it cures. Control joints, which are commonly used in exterior decking to control such cracking, are normally not used in interior slabs. The reinforcement recommended above is intended to reduce cracking and its proper placement is critical to the performance of the slab. The minor shrinkage cracks, which often form in interior slabs, generally do not present a problem when carpeting, linoleum, or wood floor coverings are used. The slab cracks can, however, lead to surface cracks in brittle floor coverings such as ceramic tile.

EXTERIOR CONCRETE DECKS

Exterior concrete decking should be cast over firm undisturbed beach sand or approved compacted fill and be reinforced with a minimum of #3 bars placed 18 inches on center, each way. Decking that caps a retaining wall should be provided with a flexible joint to allow for the normal one to two percent deflection of the retaining wall. Decking that does not cap a retaining wall should not be tied to the wall. The space between the wall and the deck will require periodic caulking to prevent moisture intrusion into the retaining wall backfill. The subgrade should be moistened prior to placing concrete.

CEMENT TYPE AND CORROSION PROTECTION

A representative sample of the onsite soil was obtained during field exploration for laboratory testing. Corrosion test results are included in Appendix I. The results indicate that concrete structures in contact with the onsite soils will have negligible exposure to water-soluble sulfates.

The results of the laboratory testing also indicate that the near-surface soil onsite is considered corrosive to ferrous metals. The corrosion information presented in Appendix I of this report should be provided to the underground utility subcontractor.

DRAINAGE

Control of site drainage is important for the performance of the proposed project. Pad and roof drainage should be collected and transferred to the street or approved location in non-erosive drainage devices. Drainage should not be allowed to pond on the pad or against any foundation or retaining wall. Drainage control devices require periodic cleaning, testing, and maintenance to remain effective.

Infiltration Pit

Typically, infiltration systems are utilized in areas underlain by pervious granular earth materials that have high percolation characteristics. In addition, infiltration systems are normally planned at least 10 feet from adjacent property lines or public right-of-way, and 15 feet from a 1:1 plane projected from the bottom of adjacent structural foundations. Since the site is mapped within a liquefaction-designated zone and the groundwater level is just below the surface, infiltration pits are not recommended on the subject site.

As an alternative, a biofiltration system may be installed on the site in accordance with the County of Los Angeles Low Impact Development Standards Manual (County of Los Angeles, 2009). A planter box may be used to capture and treat storm-water runoff through various soil layers before discharging water to the street storm drain. Planter boxes should be an impermeable structure that is equipped with an underdrain to prevent water infiltration to the underlying subsurface earth materials. Planter boxes may be situated above ground and placed adjacent to buildings. Planter boxes should be designed as freestanding and for an inward equivalent fluid pressure of 43 pounds-per-cubic-foot. This fluid pressure includes possible vehicular surcharge. Byer Geotechnical, Inc., should be provided with the final plans to verify the location of the planter boxes.

WATERPROOFING

Interior and exterior retaining walls are subject to moisture intrusion, seepage, and leakage, and should be waterproofed. Waterproofing paints, compounds, or sheeting can be effective if properly installed. Landscape areas above the wall should be sealed or properly drained to prevent moisture contact with the wall or saturation of wall backfill.

PLAN REVIEW

Formal plans ready for submittal to the building department should be reviewed by Byer Geotechnical. Any change in scope of the project may require additional work.

SITE OBSERVATIONS DURING CONSTRUCTION

The building department requires that the geotechnical engineer provide site observations during grading and construction. Foundation excavations should be observed and approved by the geotechnical engineer or geologist prior to placing steel, forms, or concrete. The engineer/geologist should observe bottoms for fill, compaction of fill, temporary excavations and shoring. All fill that is placed should be approved by the geotechnical engineer and the building department prior to use for support of structural footings and floor slabs.

Please advise Byer Geotechnical, Inc., at least 24 hours prior to any required site visit. The building department stamped plans, the permits, and the geotechnical reports should be at the job site and available to our representative. The project consultant will perform the observation and post a notice at the job site with the findings. This notice should be given to the agency inspector.

FINAL REPORTS

The geotechnical engineer will prepare interim and final compaction reports upon request. The geologist will prepare reports summarizing pile excavations.

CONSTRUCTION SITE MAINTENANCE

It is the responsibility of the contractor to maintain a safe construction site. The area should be fenced and warning signs posted. All excavations must be covered and secured. Soil generated by foundation excavations should be either removed from the site or placed as compacted fill. Soil should not be spilled over any descending slope. Workers should not be allowed to enter any unshored trench excavations over five feet deep. Water shall not be allowed to saturate open footing trenches.

GENERAL CONDITIONS AND NOTICE

This report and the exploration are subject to the following conditions. Please read this section carefully; it limits our liability.

In the event of any changes in the design or location of any structure, as outlined in this report, the conclusions and recommendations contained herein may not be considered valid unless the changes are reviewed by Byer Geotechnical, Inc., and the conclusions and recommendations are modified or reaffirmed after such review.

The subsurface conditions, excavation characteristics, and geologic structure described herein have been projected from test excavations on the site and may not reflect any variations that occur between these test excavations or that may result from changes in subsurface conditions.

Fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, irrigation, and other factors not evident at the time of the measurements reported herein. Fluctuations also may occur across the site. High groundwater levels can be extremely hazardous. Saturation of earth materials can cause subsidence or slippage of the site.

If conditions encountered during construction appear to differ from those disclosed herein, notify us immediately so we may consider the need for modifications. Compliance with the design concepts, specifications, and recommendations requires the review of the engineering geologist and geotechnical engineer during the course of construction.

THE EXPLORATION WAS PERFORMED ONLY ON A PORTION OF THE SITE, AND CANNOT BE CONSIDERED AS INDICATIVE OF THE PORTIONS OF THE SITE NOT EXPLORED.

This report, issued and made for the sole use and benefit of the client, is not transferable. Any liability in connection herewith shall not exceed the Phase I fee for the exploration and report or a negotiated fee per the Agreement. No warranty is expressed, implied, or intended in connection with the exploration performed or by the furnishing of this report.

THIS REPORT WAS PREPARED ON THE BASIS OF THE PRELIMINARY DEVELOPMENT PLAN FURNISHED. FINAL PLANS SHOULD BE REVIEWED BY THIS OFFICE AS ADDITIONAL GEOTECHNICAL WORK MAY BE REQUIRED.

Byer Geotechnical appreciates the opportunity to provide our service on this project. Any questions concerning the data or interpretation of this report should be directed to the undersigned.

Respectfully submitted,
BYER GEOTECHNICAL, INC.

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Enc: List of References (2 Pages)
- Appendix I - Laboratory Testing and Log of Borings
  - Laboratory Testing (2 Pages)
  - Shear Test Diagrams (2 Pages)
  - Consolidation Curves (9 Pages)
  - Log of Borings 1 - 3 (7 Pages)
- Appendix II - Seismic Calculations
  - Seismic Sources (2 Pages)
  - Seismic Hazard Deaggregation Chart
  - Interpretation of Electronic Piezocone (CPT) Data (10 Pages)
  - Liquefaction Susceptibility Analysis: SPT Method (2 Page/Sheet)
  - Liquefaction Susceptibility Analysis: CPT Method (5 Sheets)
- Appendix III - Geotechnical Calculations and Figures
  - Pile Axial Capacity Charts - Calculation Sheet #1 and #2 (2 Pages)
  - Retaining Wall Calculation Sheets #3 - #6 (4 Pages)
  - Sheet Pile Wall Pressure Distribution Charts - Calculation Sheets #7 and #8 (2 Pages)
  - Aerial Vicinity Map
  - Regional Topographic Map
  - Regional Geologic Map
  - Regional Fault Map
  - Seismic Hazard Zones Map
  - Tsunami Inundation Map
  - Sections A and B (2 Sheets)
  - Site Plan

xc: (4) Addressee (E-mail and Mail)

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Software

EZ-FRISK 7.62, Risk Engineering, Inc.

APPENDIX I

Laboratory Testing and Log of Borings