CHRISTIAN WHEELER
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REPORT OF PRELIMINARY GEOTECHNICAL INVESTIGATION

PROPOSED SINGLE-FAMILY RESIDENCE

ASSESSOR'S PARCEL NUMBER 258-173-03

4th STREET

ENCINITAS, CALIFORNIA

SUBMITTED TO

MR. MARK MCKENNA

3137 DUSTY HILLS TRAIL

ENCINITAS, CALIFORNIA 92024

SUBMITTED BY

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July 6, 2009

Mark McKenna
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CWE 2070453.05

SUBJECT: REPORT OF PRELIMINARY GEOTECHNICAL INVESTIGATION, PROPOSED SINGLE-FAMILY RESIDENCE, ASSESSOR'S PARCEL NUMBER 258-173-03, 4th STREET, ENCINITAS, CALIFORNIA

Dear Mr. McKenna:

In accordance with your request and our proposal dated May 14, 2009, we have completed a preliminary geotechnical investigation for the subject project. We are presenting herewith a report of our findings and recommendations.

In general, our findings indicate that the subject property is suitable for the proposed single-family residence and associated improvements, provided the recommendations provided herein are followed. Based on our previous subsurface explorations on the subject site and the adjacent lot to the south, as well as our recent geotechnical observations made during the infilling of a previously existing swimming pool on-site, the site has been determined to be underlain by Tertiary-age and Quaternary-age formational deposits that are overlain by a thin and irregular veneer of artificial fill. Based on our investigation and the results of our quantitative bluff stability and erosion analyses, the proposed residence could be sited landward of a 40-foot coastal bluff top setback, provided a series of shear pins is placed along the western edge of the proposed residence. Specific recommendations pertaining to the construction of such shear pins, which would serve to increase the gross stability of the site, are included herein along with our findings, conclusions, and recommendations for the proposed residence.

Other than issues pertaining to the presence of the coastal bluff along the west side of the site, the property is located in an area that is relatively free of other geologic hazards that will have a significant effect on the proposed construction. The most likely geologic hazard, other than bluff stability or erosion, that could affect the site is ground shaking due to seismic activity along one of the regional active faults. However, construction in accordance with the requirements of the 2007 edition of the California Building Code and the local governmental agencies should provide a level of life-safety suitable for the type of development proposed.

If you have any questions after reviewing this report, please do not hesitate to contact our office. This opportunity to be of professional service is sincerely appreciated.

Respectfully submitted,

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PROPOSED SINGLE-FAMILY RESIDENCE

ASSESSOR'S PARCEL NUMBER 258-173-03

4th STREET

ENCINITAS, CALIFORNIA

INTRODUCTION AND PROJECT DESCRIPTION

This report presents the results of a preliminary geotechnical investigation performed for a proposed single-family residence to be constructed at the residential lot identified as Assessor's Parcel Number 258-173-03, and located along the west side of 4th Street, in the city of Encinitas, California. Figure Number 1 presented on the following page provides a vicinity map showing the location of the property.

The subject site consists of a rectangular-shaped residential lot located adjacent to and west of Fourth Street in the city of Encinitas, and is identified as Assessor's Parcel Number 258-173-03. The site has approximately 100 feet of frontage along Fourth Street and is approximately 240 to 250 feet deep. The site previously supported a relatively shallow swimming pool and associated hardscape areas that were removed in November 2008. The backfilling of the pool excavation was performed under the observation and testing of this firm. The topography of the site is characterized by a relatively level pad that is at an elevation of approximately 105 feet and on which the pool and other improvements were previously located, and a steep coastal bluff that descends to the beach below on the western portion of the site. The edge of this bluff is about 80 to 100 feet west of the front, eastern property line of the site.

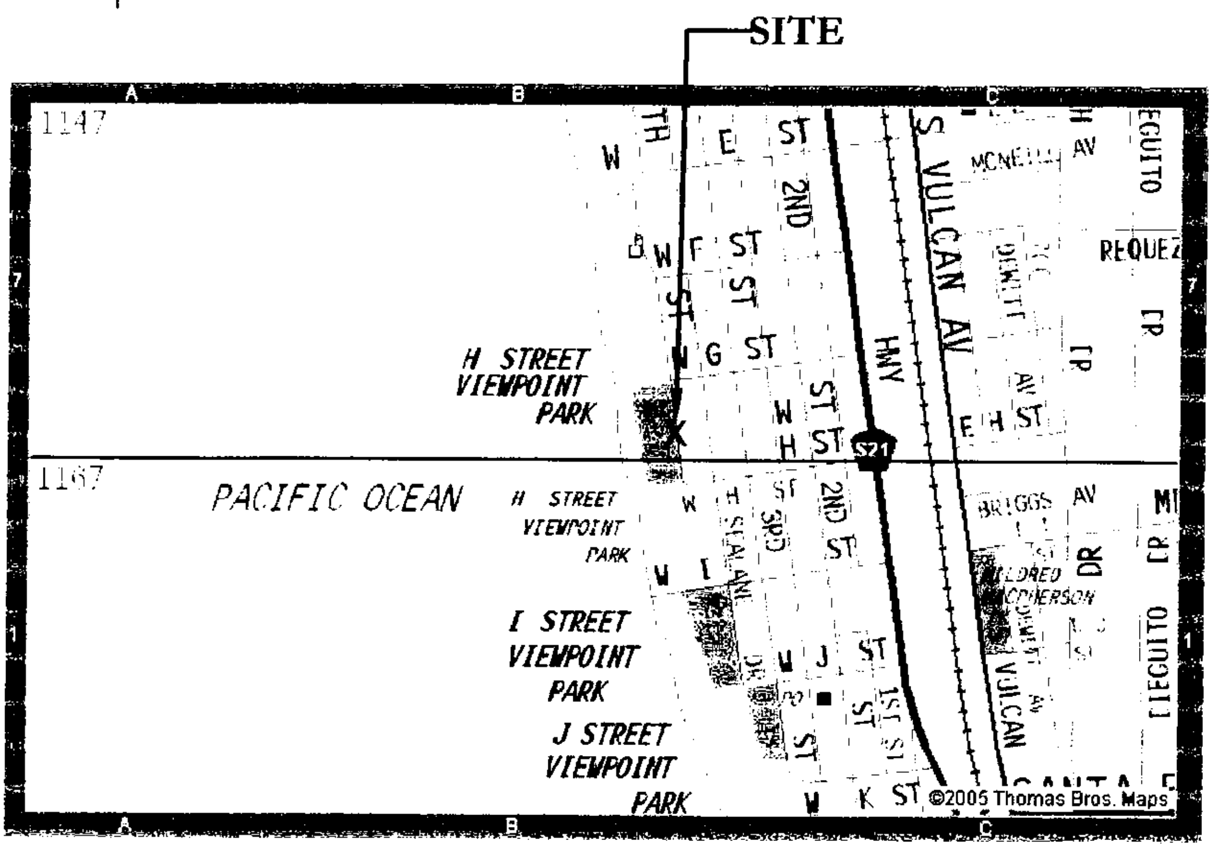
Although development plans are still in the conceptual stage, we understand that a new, two-story, single-family residence is proposed to be constructed on the lot. We anticipate that the new residence will incorporate a subterranean basement and/or parking level and that the new residence and improvements will be supported by a conventional shallow foundation system and cast-in-place concrete piers that will also serve as shear pins along the west side of the residence. The subterranean portions of the residence are expected to be of concrete masonry construction while the above-grade portions of the residence are expected to be of wood-frame construction with possibly some steel moment frames. Grading to accommodate the new structure is anticipated to consist of cuts of up to approximately ten feet from existing grades.

To aid in the preparation of this report, we were provided with an undated topographic map of the subject site and adjacent areas prepared by Di Donato Associates. It is our understanding that the topographic data

SITE VICINITY MAP

(Adapted from Thomas Brothers Maps)

PROPOSED SINGLE-FAMILY RESIDENCE
A.P.N. 258-173-03
4TH STREET
ENCINITAS, CALIFORNIA



CWE 2070453.05 | July 6, 2009 | Figure 1 presented on this plan was collected in 2007. A copy of this plan was used as a base map for our Site Plan and Geotechnical Map, and is included herein as Plate No. 1. In addition, three geologic cross-sections of the site, in its conceptually proposed configuration, are presented as Plate Nos. 2 through 4.

This report has been prepared for the exclusive use of Mark McKenna and his design consultants for specific application to the project described herein. Should the project be changed in any way, the modified plans should be submitted to Christian Wheeler Engineering for review to determine their conformance with our recommendations and to determine if any additional subsurface investigation, laboratory testing and/or recommendations are necessary. Our professional services have been performed, our findings obtained and our recommendations prepared in accordance with generally accepted engineering principles and practices. This warranty is in lieu of all other warranties, express or implied.

PROJECT SCOPE

Our firm has previously investigated the subject site and the adjacent residential lot to the south identified as Assessor's Parcel Number 258-173-04 (CWE, 2008). Our geotechnical investigation of the subject lot and adjacent parcel to the south consisted of surface reconnaissance, subsurface exploration, obtaining representative soil samples, laboratory testing, analysis of the field and laboratory data, and review of relevant geologic literature. The scope of services included in the preparation of this report of preliminary geotechnical investigation for the subject residence incorporated the findings of our previous geotechnical investigation and analysis performed for the subject lot and adjacent lot. Our investigation consisted of: surface reconnaissance, subsurface explorations, obtaining representative disturbed and relatively undisturbed samples, laboratory testing, analysis of the field and laboratory data, research of available geologic literature and geotechnical documents pertaining to the site, and preparation of this report. More specifically, the intent of this investigation was to:

a) Evaluate the subsurface conditions of the site to the depths influenced by the proposed construction based on the previous explorations we made on and adjacent to the site;

b) Evaluate, by our previously performed laboratory tests and experience with similar soil types, the general engineering properties of the various strata that may influence the proposed construction, including bearing capacities, expansive characteristics and settlement potential;

c) Describe the general geology at the site, including possible geologic hazards that could have an effect on the proposed construction, and provide the seismic design parameters as required by the 2007 edition of the California Building Code;

d) Determine the location of the top of the coastal bluff on site;

e) Perform a series of computer-assisted slope stability analyses of the bluff in order to determine the required setback from the edge of the bluff;

f) Provide recommendations for the installation of shear pins to increase the gross stability of the subject site;

g) Determine the mean annual rate of bluff top recession at the site;

h) Recommend an appropriate foundation system for the type of structure anticipated and develop soil engineering criteria for the recommended foundation design;

i) Present our professional opinions in a report, which will include in addition to our conclusions and recommendations, a plot plan, logs of the borings previously drilled on and adjacent to the site and a summary of the previously performed laboratory test results.

Our scopes of services did not include assessment of hazardous substance contamination, recommendations to prevent floor slab moisture intrusion or the formation of mold within the proposed residence, or any other services not specifically described herein. Although a test for the presence of soluble sulfates within the soils that may be in contact with reinforced concrete was performed as part of the scope of our services for our previous work on the subject site and adjacent lot, it should be understood Christian Wheeler Engineering does not practice corrosion engineering. If such an analysis is considered necessary, we recommend that the client retain an engineering firm that specializes in this field to consult with them on this matter. The results of these tests should only be used as a guideline to determine if additional testing and analysis is necessary.

FINDINGS

SITE DESCRIPTION

The subject site is a nearly rectangular-shaped parcel of land identified as Assessor's Parcel Number 258-173-03, located in the City of Encinitas, California. The site, which currently supports a few small site walls, a shed, hardscape areas, and fences, is bounded to the east by 4th Street, to the west by the Pacific Ocean, and to the north and south by developed residential properties. The site previously supported an approximately 6-foot-deep swimming pool that was removed in November 2008. Our firm provided observation and testing of the structural fill placed in the excavation created when the pool was removed (CWE, 2008). The site has approximately 100 feet of frontage along Fourth Street and is approximately 240 to 250 feet deep. The front, eastern portion of the site is relatively level and supports the existing improvements. From the eastern perimeter of the site, this relatively level area extends westerly approximately 80 to 100 feet to the top of bluff. The western, approximately two-thirds of the property is comprised of a coastal bluff that is about 100 feet in height. The bluff descends to the beach area and the Pacific Ocean.

Topographically, the site ranges from approximately 8 feet to 110 feet above Mean Sea Level (MSL), with elevations landward of the top of bluff ranging from approximately 101 feet to 110 feet MSL. Vegetation across the upper, relatively level portions of the site consists of typical residential landscaping, including grasses, shrubs, and small to medium-sized trees. Vegetation across the undeveloped, coastal bluff portions of the site is limited to scattered light grasses along the upper and middle bluff areas.

GENERAL GEOLOGY AND SUBSURFACE CONDITIONS

GEOLOGIC SETTING AND SOIL DESCRIPTION: The subject site is located in the Coastal Plains Physiographic Province of San Diego County. Based on our investigation, we have determined that the site is generally underlain by Tertiary-age deposits of the Delmar Formation, mantled by Quaternary-age terrace deposits, with a relatively thin veneer of artificial fill at the surface. The materials encountered within our subsurface exploration are described below in order of increasing age:

> **ARTIFICIAL FILL (Qafu/Qafd):** An approximately 2- to 4-foot-thick layer of undocumented fill material (Qafu) was encountered within our borings drill on the upper, relatively level portion of the subject site and adjacent lot to the south. Additionally, the removal of the above described, previously existing swimming pool on-site resulted in the placement of up to about 7 feet of documented fill (Qafd) within the southern portion of the upper, relatively level area of the site. The

approximate limits of the infilled swimming pool excavation are presented on Plate No. 1 of this report. The fill encountered within our borings generally consist of medium brown and medium grayish-brown, silty sand (SM) that was typically moist and loose to medium dense in consistency. The fills encountered within our borings are expected to possess a low expansion index and a moderate settlement potential in their present condition. The fill placed within the excavation created by removing the swimming pool generally consist of light brown, silty sand (SM) with varying amounts of gravel. This fill was compacted to a minimum relative compaction of 90% and was noted to be typically moist and dense in consistency. The structural fills placed under our observation and testing (CWE, 2008 b) are expected to possess a low expansion index and a low settlement potential in their present condition.

TERRACE DEPOSITS (Qt_u & Qt_l): Quaternary-age marine terrace deposits were encountered below the fill material within each of our borings and within the base of the excavation created during the removal of the swimming pool. The terrace deposits are also visible along the mid and upper portions of the coastal bluff face. These terrace deposits can be further classified as having distinct "Upper" and "Lower" units. Specifically, the upper terrace deposits were measured to have a thickness of about 8 to 10 feet across the site and were noted to generally consisted of light to medium reddish-brown and medium brown, poorly-graded sand-silty sand (SP-SM) that was moist and medium dense in consistency. The base of the upper terrace unit is at approximate elevations of 90 feet to 95 feet above mean sea level (MSL).

Beneath the "Upper" terrace deposits, the "Lower" terrace deposits were measured to have a thickness of about 50 feet and to extend down to an elevation of about 40 feet (MSL) beneath the subject site. The lower terrace deposits generally consisted of light brown, poorly-graded sand-silty sand (SP-SM) that was moist and dense to very dense in consistency. The terrace deposits, although friable and subject to episodes of high rates of sub-aerial erosion, are considered to have moderate strength parameters with regards to deep-seated bluff failure.

DELMAR FORMATION (Td): Tertiary-age sedimentary deposits of the Delmar Formation were encountered below the lower terrace deposits within our boring B-1 (drilled adjacent to the southeast portion of the subject site), and are expected to underlie the site at depth. This formational material was noted in our boring B-1 and observed along the bluff face to consist predominantly of medium greenish-gray to light yellowish-brown, silty sand (SM) that was very dense in consistency. The uppermost portions of the Delmar Formation were noted to be wet to saturated while the lower portions of these materials observed in our boring were found to be wet to moist. It should, however, be recognized that although not encountered in our deep boring drilled on-site, weaker zones of claystones and siltstone are common within this formation. The materials of the Delmar Formation are considered to possess relatively high strength parameters with regards to bluff stability.

GEOLOGIC STRUCTURE: The bedding of the materials of the Delmar Formation beneath the site was measured along the bluff face to strike from between N34°E to N58°E and dip towards the northwest at approximately 3°. Such bedding orientations, which correlate with the data presented on the geologic map of the Encinitas and Rancho Santa Fe 7½-minute quadrangles (CDMG, 1996), display a slight out-of-slope component along the face of the coastal bluff on-site with apparent dip angles of about 2° out-of-slope along each of our geologic cross sections. Based on our experience within the vicinity of the site, the erosional contact between the generally massive terrace deposits and the underlying materials of the Delmar Formation is expected to dip up to a few degrees (<4°) to the southwest across the subject site and surrounding areas.

BLUFF EDGE: Based on available information, the edge of the bluff is located along the top of the west side of the upper, relatively level portion of the site at elevations of 109 feet to 110 feet (MSL). The edge of the bluff (for development purposes) is defined in the City of Encinitas Code as *"the upper termination of a bluff. When the top edge of the bluff is rounded away from the face of the bluff as a result of erosional processes related to the presence of the steep bluff face, the edge shall be defined as that point nearest the bluff beyond which the downward gradient of the land surface increases more or less continuously until it reaches the general gradient of the bluff."* Our interpretation of the approximate "edge of the bluff", based on our on-site reconnaissance and a review of the aerial photographs, is shown on both the Site Plan and Geotechnical Map presented as Plate No. 1 and on the geologic cross-sections presented on Plate Nos. 2-4.

GROUNDWATER: Groundwater was encountered within our boring B-1 at a depth of about 63½ feet below the existing ground surface. This depth corresponds to an approximate elevation of 40 feet (MSL). It should be recognized that, similar to other areas of the Encinitas coastline, the groundwater encountered within our subsurface exploration (B-1) and observed to be seeping out the bluff face, is considered to be water that is perched along the geologic contact between the terrace deposits (lower unit) and the underlying, less permeable materials of the Delmar Formation. This perched groundwater, along with a lower, free groundwater table, have been modeled in our analyses of the bluff's stability.

TECTONIC SETTING: No faults are known to traverse the subject site. However, it should be noted that much of Southern California, including the San Diego County area, is characterized by a series of Quaternary-age fault zones that consist of several individual, en echelon faults that generally strike in a northerly to northwesterly direction. Some of these fault zones (and the individual faults within the zone) are classified as "active" according to the criteria of the California Division of Mines and Geology. Active fault zones are those that have shown conclusive evidence of faulting during the Holocene Epoch (the most recent 11,000 years). The Division of Mines and Geology used the term "potentially active" on Earthquake Fault Zone maps until 1988 to refer to all Quaternary-age (last 1.6 million years) faults for the purpose of evaluation for possible zonation in accordance with the Alquist-Priolo Earthquake Fault Zoning Act and identified all Quaternary-age faults as "potentially active" except for certain faults that were presumed to be inactive based on direct geologic evidence of inactivity during all of Holocene time or longer. Some faults considered to be "potentially active" would be considered to be "active" but lack specific criteria used by the State Geologist, such as *sufficiently active* and *well-defined*. Faults older than Quaternary-age are not specifically defined in Special Publication 42, Fault Rupture Hazard Zones in California, published by the California Division of Mines and Geology. However, it is generally accepted that faults showing no movement during the Quaternary period may be considered to be "inactive".

A review of available geologic maps indicates that the active Rose Canyon Fault Zone is located approximately 2.4 miles west of the subject site. Other active fault zones in the region that could possibly affect the site include the Newport-Inglewood and Palos Verdes Fault Zones to the northwest, the Coronado Bank Fault Zone to the west, and the Elsinore and Earthquake Valley Fault Zones to the northeast.

The following Table I presents the active faults that are considered most likely to significantly affect the site over the anticipated economic lifetime of the proposed residence.

TABLE I: PROXIMAL FAULT ZONES

Fault Zone	Distance	Max. Magnitude Earthquake
Rose Canyon	2.4 miles	7.2 Magnitude
Newport-Inglewood	11 miles	7.1 Magnitude
Coronado Bank	17 miles	7.6 Magnitude
Elsinore	28 miles	7.1 Magnitude
Palos Verdes	41 miles	7.2 Magnitude
Earthquake Valley	43 miles	6.5 Magnitude

GEOLOGIC HAZARDS

GENERAL: No geologic hazards of sufficient magnitude to preclude the proposed residential use of the site as we presently understand it are known to exist. In our professional opinion and to the best of our knowledge, the site is suitable for the proposed development provided measures are taken to increase the global stability of the coastal bluff on-site.

SEISMIC DESIGN FACTORS: A likely geologic hazard to affect the site is ground shaking as a result of movement along one of the major active fault zones mentioned above. The fault most likely to have a significant effect on the site is the Rose Canyon Fault, located about 4 kilometers southwest of the site. The seismic design factors applicable to the subject site are provided below. The seismic design factors were determined in accordance with the 2007 California Building Code. The site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters are presented below:

TABLE III: SEISMIC DESIGN FACTORS

Site Coordinates: Latitude Longitude	33.0412° -117.2963°
Site Class	C
Site Coefficient F_a	1.0
Site Coefficient F_v	1.3
Spectral Response Acceleration at Short Periods S_s	1.430 g
Spectral Response Acceleration at 1 Second Period S_1	0.540 g
$S_{MS}=F_aS_s$	1.430 g
$S_{M1}=F_vS_1$	0.702 g
$S_{DS}=2/3*S_{MS}$	0.954 g
$S_{D1}=2/3*S_{M1}$	0.468 g

Probable ground shaking levels at the site could range from slight to moderate, depending on such factors as the magnitude of the seismic event and the distance to the epicenter. It is likely that the site will experience the effects of at least one moderate to large earthquake during the life of the proposed improvements

BLUFF EROSION: Coastal bluff recession is a process that is presently occurring in much of coastal San Diego County. Typically, coastal recession occurs through three modes that include: 1) undercutting of the base of the cliff by wave action and subsequent block falls of the overlying materials; 2) undercutting of the terrace deposits or other surficial material, initiated by water seepage conditions at the formational contact, and subsequent slumping of the overlying materials; and 3) deep-seated rotational-type failures.

The mode of historical recession at the subject site proper and in the immediate vicinity appears to be manifested both as small block falls caused by erosion along the fractures and joints in the Delmar Formation and by subaerial erosion of the overlying terrace deposits caused by severe storm conditions and/or adverse drainage conditions. The rate of erosion is variable with periods of very little recession alternating with episodes in which a block of the Delmar Formation falls from the face of the seacliff or substantial surficial erosion occurs. Evidence of some relatively recent, small blockfalls from the face of the cliff was observed just to the south of the site during our site reconnaissance. Based on the available information, it appears that the overall recession rate of the Delmar Formation in the area of the subject site is approximately one-half inch to one inch per year and that the recession rate of the terrace deposits and other surficial materials ranges from less than an inch per year to several feet per year with an average rate of a few inches to several inches per year in unprotected areas.

The Shoreline Erosion Assessment and Atlas of the San Diego Region prepared by the California Department of Boating and Waterways and San Diego Association of Governments in 1994 indicates that the shoreline risk at the site is high due to unfavorable geology, inadequate setback, and a narrow beach. The "unfavorable geology" condition is defined in the Shoreline Erosion Assessment and Atlas of the San Diego Region by the presence of a relatively low bedrock cliff (approximately 35 feet) and a relatively thick section (approximately 55 feet) of unconsolidated terrace deposits. The Shoreline Atlas indicates that no man-made shoreline protection is evident along this section of the shoreline.

In order to quantify the rate of bluff top retreat of the exiting coastal bluff adjacent to the subject site we have obtained and reviewed copies of the County of San Diego aerial photographs, Packet 37, D1 and D2, which were taken in 1928. These photographs were enlarged to an approximate scale of 1 inch to 430 feet. Our estimation of the approximate scale of the enlarged photographs was performed by scaling the distances between previous and existing street locations on the enlarged aerial photographs (both D1 and D2) and recent County of San Diego 200-scale topographic maps.

Utilizing the previous and existing location of the rail line that exists approximately 1155 to 1175 feet to the east of the subject site, we estimate that that in 1928 the edge of the bluff top to was located approximately 103 feet to the west of the eastern perimeter of the subject site along geologic cross section D-D'. Along cross sections A-A' and E-E', we estimate that in 1928 the edge of the bluff top was located approximately 111 feet and 115 feet to the west of the eastern perimeter of the subject site, respectively.

As presented on our revised Site Plan and Geotechnical Map, the edge of the bluff top is currently located 86 feet, 103 feet, and 101 feet to the west of the eastern perimeter of the subject site along cross sections D-D', E-E', and A-A', respectively. As such, we calculate that, along the alignment of cross sections A-A', E-E', and D-D', the edge of the bluff top has retreated approximately 10 feet, 12 feet, and 17 feet during the 79 years from 1928 to 2007, respectively. Such distances of bluff top retreat equate to approximate mean annual rates of retreat of approximately 0.13 foot/year (1½ inches/year) to 0.22 foot/year (3 inches/year). This range of mean annual bluff top retreat correlates with the modified long-term estimate of seacliff retreat presented on page 24 of the referenced City of Encinitas General Plan, Beach Bluff Erosion Technical Report, of 0.2 foot/year (Zeisler Kling, 1994). It should be noted that the modified long-term estimate of seacliff retreat presented by Zeisler Kling is based on both shelf-slope and littoral sediment lens methods of analysis.

As described in detail in the referenced Beach Bluff Erosion Technical Report prepared by Zeiser Kling Consultants, Inc., historically, bluff top and sea cliff retreat within the area of the subject site begins with the weakening of the lower bluff area by wave attack and, to a much lesser degree over the lifetime of any particular development, sea level rise. Such weakening of the lower bluff area results in the formation of undercut surfaces and joints that, over time, lead to rockfall and block failure within the lower bluff face. Following such failures within the lower bluff face, the middle portions of the bluff face begin to slump off and fall to the beach bench, covering any talus that may remain at the base of the bluff. As this process continues, subsequent rotational failures and erosional processes erode the middle and upper portions of the bluff face. Such rotational failures and erosional processes continue to a point where the over-steepened, upper and middle portions of the bluff face reach an inclination controlled by the angle of internal friction of the terrace deposits that comprise the middle to upper bluff face. At some point, once all of the talus has been removed from the base of the bluff face, the entire process is repeated.

During our recent visits to the subject site and along the base of the bluff face adjacent to the subject site we noted that portions of the lower bluff face to the south of the subject site have recently experienced block failures. Such recent failures have resulted in a few feet of regression of the lower bluff face to the south of the subject site.

In consideration of the apparently recent block failures along the bluff face to the south of the subject site and the nearly parallel to bluff jointing within the Delmar Formation across the area of the subject site, similar block failures may be anticipated along the lower bluff area adjacent to the subject site within the design life of the proposed project. In consideration of this, our bluff erosion plane analysis has been performed assuming that the lower bluff face adjacent to the subject site is 4 feet to the east of its current location. Furthermore, as considered prudent, we have incorporated the highest of our calculated mean annual rates of bluff top retreat of 0.22 foot/year in our bluff erosion plane analysis. Considering this rate and the likelihood of block failure within the lower bluff face on-site in the not too distant future, based on observations made on the adjacent site to the south, we anticipate that over the design life of the proposed residence (assumed to be 75 years), the bluff face adjacent to the subject site may retreat approximately 21 feet. The location of the daylight line of our bluff erosion plane analysis is presented on our revised Site Plan and Geotechnical Map and geologic cross sections included herein as Plate Nos. 1 through 4.

It should be realized that our bluff erosion plane analysis has been performed utilizing a mean annual rate of bluff top retreat that is slightly in excess of the modified long-term estimate of seacliff retreat presented by Zeisler Kling. Furthermore, it should be understood that both the mean annual rate of bluff top retreat utilized in our bluff erosion plane analysis and the modified long-term estimate of seacliff retreat presented by Zeisler Kling represent average rates of bluff top/sea cliff retreat. As such, year to year variations in the rate of bluff top recession should not only be anticipated but also expected.

LIQUEFACTION: The native materials at the site are not subject to liquefaction due to such factors as soil density, grain-size distribution, and the absence of shallow groundwater conditions.

FLOODING: The site is located outside the boundaries of both the 100-year and the 500-year floodplains according to the maps prepared by the Federal Emergency Management Agency.

TSUNAMIS: Tsunamis are great sea waves produced by a submarine earthquake or volcanic eruption. Historically, the San Diego area has been free of tsunami-related hazards and tsunamis reaching San Diego have generally been well within the normal tidal range. It is thought that the wide continental margin off the coast acts to diffuse and reflect the wave energy of remotely generated tsunamis. The largest historical tsunami to reach San Diego's coast was 4.6 feet high, generated by the 1960 earthquake in Chile. A lack of knowledge about the offshore fault systems makes it difficult to assess the risk due to locally generated tsunamis. However, due to the site's elevation, the portion of the site to be developed is considered to possess a low risk potential from tsunamis.

SEICHES: Seiches are periodic oscillations in large bodies of water such as lakes, harbors, bays or reservoirs. Due to the site's location, it will not be affected by seiches.

LANDSLIDE POTENTIAL AND BLUFF STABILITY

GENERAL: The bluff top site is in an area that is considered most susceptible to slope stability hazards due to such factors as the character of the geologic units, the presence of fractures or other planes of weakness, the steepness of the bluff, erosion of the bluff face by surface runoff down the face of the bluff, and wave action along the base of the bluff.

The Relative Landslide Susceptibility and Landslide Distribution Map of the Encinitas Quadrangle prepared by the California Division of Mines and Geology indicates that the relatively level, upper, easternmost portions of the site are situated within Relative Landslide Susceptibility Area 2. Area 2 is considered to be "marginally susceptible" to slope failures. The bluff area within the central and western portions of the site, however, is situated within Relative Landslide Susceptibility Area 4-1. Area 4 is considered to be "most susceptible" to slope failures; Subarea 4-1 includes slopes considered to be generally outside the boundaries of known landslides, but which contain observably unstable slopes that are underlain by often weak materials and unfavorable geologic structure. Based on our investigation, the site was found to be underlain at shallow depths by moderately friable terrace deposits over dense, well-consolidated, sandstones of the Delmar Formation.

GROSS STABILITY ANALYSIS: To analyze the stability of the site and existing bluff, three cross-sections, the locations of which are shown on Plate No. 1, were drawn perpendicular to the bluff. These cross-sections, labeled as A-A', D-D', and E-E' are included herein as Plate Nos. 2 through 4 and are also presented with the plots of the stability analyses in Appendices B through D. Although no site development plans have been provided to us for review, based on our conversations with the project architect, we prepared cross sections A-A', D-D', and E-E' to reflect the inclusion of a subterranean level of the proposed residence at an approximate elevation of 99 feet (MSL). Cross sections labeled B-B' and C-C' were not included in this analysis as they were previously prepared to represent the existing conditions on the residential lot to the south of the subject site.

As described above in the "Geologic Setting and Soil Description" section of this report, the existing bluff is comprised of Tertiary-age material of the Delmar Formation that is overlain by a relatively thick layer of Quaternary-age terrace deposits. A thin and irregular veneer of man-placed fill soils overlies the terrace deposits along the eastern portion of the site.

As required by City staff, we have completed a series of bluff failure plane analyses, which includes analyses of the existing bluff modeling static and pseudo-static conditions. The analyses and discussion address bluff failures originating along both the lower bluff face and the mid to upper bluff areas. Our stability analyses address the bluff stability for both rotational and block failures.

The minimum factors-of-safety that are considered stable are 1.5 for static slope (bluff) stability analyses and 1.1 for pseudo-static slope (bluff) stability analyses. Our bluff stability analyses and our identification of the lines behind the top of bluff that depict minimum static and pseudo-static factors-of-safety of 1.5 and 1.1, respectively, have been performed incorporating modes of failure observed during both historic and on-going bluff failures (Zeiser Kling, 1994). It should be recognized that lines depicting the minimum static and pseudo-static factors-of-safety of 1.5 and 1.1, respectively, have been plotted on Plate No. 1 to represent current and proposed (including a subterranean level of the proposed residence) site conditions.

No preemptive measures, as defined by Section 30.34.020 C of the City's Municipal Code (item #2), are known to currently exist at the subject site. However, it should be recognized that the City may consider the proposed shear pins to be a preemptive measure.

STRENGTH PARAMETERS: The strength parameters for the materials comprising the bluff were estimated based on the results of direct shear testing and our experience with similar soil types in the vicinity of the site. The results of our direct shear testing are presented in Appendix A of this report. Additionally, in consideration of the slight out-of-slope bedding orientations of the materials of the Delmar Formation along the lower portions of the bluff, our stability analyses that modeled block-type failures have been performed incorporating anisotropic soil strength parameters for the Delmar Formation. Specifically, a 50% reduction in the shear strength parameters of the Delmar Formation have been modeled in our analyses from 0° to the maximum encountered apparent dip of 2° from the horizontal. It should be noted that such a reduction in strength parameters to model along bedding conditions is rather conservative. However, the results of our circular-type failure analyses (not the block-type analyses) governed our plotting of the 1.5 static and 1.1 pseudo-static factor-of-safety lines along the top of bluff. Plots of the anisotropic soil definition of the Delmar Formation modeled in our block-type analyses are presented at the rears of Appendices A through D.

The unit weights of the earth materials that underlie the subject site and adjacent areas utilized in our stability analyses were chosen based on the results of our laboratory testing and our experience with similar materials in the vicinity of the subject site. It is our professional opinion that the strength parameters and unit weights presented below and utilized in our stability analyses provide for conservative slope stability analyses.

Soil Type	Unit Weight, γ	Phi, ϕ	Cohesion, c
Artificial Fill (Qaf)	115 pcf	30°	100 psf
Beach Sands	100 pcf	30°	0 psf
Terrace Deposits (Upper)	115 pcf	36°	275 psf
Terrace Deposits (Lower)	115 pcf (wet)/125 pcf (saturated)	37°	300 psf
Delmar Formation*	125 pcf (wet)/130 pcf (saturated)	37°	1,500 psf

*Note: As described above, the strength parameters of the Delmar Formation were reduced by 50% (ϕ=18½° and c=750 psf) to model potential zones of weakness along bedding planes, which are common within the Delmar Formation.

METHOD OF ANALYSIS: The analyses of the gross stability of the coastal bluff along the west side of the site were performed using Version 2 of the GSTABL7© computer program developed by Garry H. Gregory, PE. The program analyzes circular, block, specified, and randomly shaped failure surfaces using the Modified Bishop, Janbu, or Spencer's Methods. The STEDwin© computer program, developed by Harald W. Van Aller, P. E., was used in conjunction with this program for data entry and graphics display.

The existing and proposed topographies of the subject site and adjacent areas along geologic cross sections A-A', D-D', and E-E' were analyzed for circular-type failures originating within both the lower and mid to upper portions of the bluff face and each failure analysis was programmed to run at least 4,000 random failure surfaces. Similarly, based on the out-of-slope bedding within the Delmar Formation along the lower bluff face, stability analyses incorporating block-type failure mechanisms, originating within the lower bluff face and also along the contact between the Delmar Formation and terrace deposits (lower) were also performed. Such analyses were also programmed to run at least 4,000 random failure surfaces. The most critical failure surfaces from each analysis were accumulated and sorted by value of the factor-of-safety. After the specified number of failure surfaces were successfully generated and analyzed, the ten most critical surfaces were plotted so that the pattern may be studied. Additionally, as required by current City standards pseudo-static stability analyses of the bluff were performed modeling each of the above-described stability analyses using kh coefficients equal to 0.15g and considering a factor-of-safety of 1.1 to be generally stable with regards to pseudo static bluff stability.

In order to model the encountered perched groundwater conditions along the contact between the Delmar Formation and terrace deposits (lower), we have included a piezometric surface within each analysis to represent the seepage along this contact. Specifically, within each analysis the earth materials three feet above and five feet below the contact between the Delmar Formation and terrace deposits were modeled to be saturated. Furthermore, free groundwater conditions were conservatively modeled within the lower portions of the Delmar Formation. The locations of the piezometric surfaces incorporated in our slope (bluff) stability analyses are presented on each of the computer plots of the results of our revised slope stability analyses (see Appendices B through D).

RESULTS OF STABILITY ANALYSIS: The results of our stability analyses indicate that the lowest, static factors-of-safety for the bluff top site in its current configuration are approximately 1.2 for circular-type failures and 1.4 for block-type failures (see Appendices B through D). Such values are less than the minimum factor-of-safety that is generally considered to be stable of 1.5 for static slope failures. Our analyses also indicate that, in its current configuration, the bluff top site demonstrates minimum, pseudo-static factors-of-safety of 0.9 for circular-type failures and 1.0 for block-type failures. Such values are less than the minimum factor-of-safety that is generally considered to be stable of 1.1 for static slope failures.

In addition to determining the minimum factors-of-safety against gross, static and pseudo-static bluff failure, we also analyzed the stability of the site, in its current configuration, to determine where on the site, in its current configuration, minimum static and pseudo-static factors-of-safety of 1.5 and 1.1, respectively, are demonstrated. As presented in Appendix B, in its current configuration, the site demonstrates minimum factors-of-safety against static and pseudo-static failures along cross section A-A' approximately 69 feet and 59 feet from the edge of the bluff, respectively. As presented in Appendix C, in its current configuration, the site demonstrates minimum factors-of-safety against static and pseudo-static failures along cross section D-D' approximately 54 feet and 42 feet from the edge of the bluff, respectively. Our analyses along cross section E-E' indicate that, in its current configuration, the site demonstrates minimum factors-of-safety against static and pseudo-static failures approximately 71 feet and 60.5 feet from the edge of the bluff, respectively (see Appendix D).

Based on our previous analysis of the stability of the bluff at the subject site and on the adjacent lot to the south, the results of our stability analyses of the existing site conditions contained herein, and our authorized scope of services, we also performed a series of stability analyses along each of the cross sections included herein to model the proposed home being sited 40 feet from the edge of the coastal bluff. As previously described, these analyses modeled a subterranean level of the home at an elevation of 99 feet (MSL). In addition, these analyses incorporated shear pins along the western side of the proposed residence to increase the gross stability of the bluff. As determined by these analyses, by modeling the proposed subterranean level (which will serve to reduce the driving forces adversely affecting the gross stability of the site) and incorporating shear pins along the west side of the proposed residence that will need to be designed to resist a force of 35,000 pounds for each linear foot of slope face, minimum static and pseudo-static factors-of safety of 1.5 and 1.1, respectively, would be demonstrated along the west side of the proposed residence set back 40 feet form the edge of the bluff. The analyses that incorporate the proposed site topographies as well as the proposed shear pins are presented in the rears of Appendices B through D.

RECOMMENDED BLUFF TOP SETBACK: Based on the results of our quantitative slope stability analyses described above, the stability conditions of the proposed site configuration, as opposed to the anticipated 75-year bluff top retreat described on pages 10 and 11 of this report, govern the recommended bluff top setback distance for future site development. Specifically, based on the results of our stability analyses and estimate of 75-year bluff top retreat, it is our professional opinion that a 40-foot bluff top setback could be applied to the proposed project provided the residence incorporate a significant subterranean level and the stability of the bluff is increased with the use of shear pins as described hereinafter.

Based on a 40-foot bluff top setback distance, our experience with similar projects in the vicinity of the subject site, and the inclusion of measures to increase the stability of the site as described herein, it is our professional opinion that the proposed residence could be constructed within areas of the site that are considered to be generally stable. Furthermore, the proposed residence and associated improvements will not adversely affect the stability of the existing bluff located along the west side of the site, provided care is taken to ensure proper drainage at the subject site, the recommendations contained herein are adhered to, and that prudent construction and maintenance procedures are followed. However, it should be recognized that we are not providing any sort of guarantee of the overall stability of the bluff face.

Our firm should be contacted to review project plans as they are developed. It should be understood that the stability analyses included herein have been based on conceptual project design ideas discussed with the project architect and yourself. As such, once plans for the proposed residence are finalized, it may be necessary to perform additional bluff stability analyses to reflect the then proposed site conditions and topographies.

In addition to the 40-foot building setback line, the daylight failure lines for both erosion and slope failures (in its current and proposed configurations) are depicted on our Site Plan and Geotechnical Map and geologic cross sections A-A', D-D', and E-E' included herein as Plate Nos. 1 though 4, respectively.

CONCLUSIONS

In general, our findings indicate that the subject property is suitable for the proposed single-family residence and associated improvements, provided the recommendations provided herein are followed. The results of our subsurface explorations indicate that the site is underlain by Tertiary-age and Quaternary-age formational deposits that are generally overlain by a thin and irregular veneer of man-placed fill soils. In our opinion, the existing fills that were not placed under our observation and testing are considered unsuitable in their present condition to support settlement-sensitive improvements. As such, all undocumented fills not removed by planned site grading will need to be removed from all portions of the site to support the proposed residence or other settlement sensitive improvements and, where necessary to achieve planned site grades, be replaced as properly compacted fill.

As required by Section 30.34.020 D of the City's Municipal Code (as amended by Ordinance 95-04 and Resolution 95-31), provided sound construction and maintenance practices are followed, including but not limited to the recommendations contained in this report and the City of Encinitas Municipal Code, it is our opinion that "the development proposed will have no adverse affect on the stability of the bluff, will not endanger life or property, and that any proposed structure or facility is expected to be reasonably safe from failure and erosion over its lifetime without having to propose any shore or bluff stabilization to protect the structure in the future."

Furthermore, as required by Section 30.34.020 D of the City's Municipal Code (as amended by Ordinance 95-04 and Resolution 95-31), the geotechnical reports for bluff top projects "shall also express a professional opinion as to whether the project can be designed or located so that it will neither be subject to nor contribute to significant geologic instability throughout the life span of the project." As such, it is our professional opinion and judgment that provided sound construction and maintenance practices are followed, including but not limited to our recommendations contained herein and the City of Encinitas Municipal Code, it is our professional opinion that the project can be designed and located so that it will neither be subject to nor contribute to significant geologic instability throughout the life span of the project.

In summary, no geologic hazards of sufficient magnitude to preclude residential use of the site as we presently understand it are known to exist. In our professional opinion and to the best of our knowledge, the site is suitable for the proposed residence and improvements. The results of our quantitative slope stability analysis indicate that, provided the site is developed in a manner consistent with the recommendations contained herein and the topographic modeling assumed in our stability analyses, the bluff will demonstrate factors-of-safety of or above the accepted minimums of 1.5 and 1.1 against static and pseudo-static slope failures at and east of the recommended 40-foot bluff top setback zone. Based on this condition, it is our opinion that the potential for a deep-seated slope failure that extends below the proposed residence is low. It should be noted, however, that this does preclude the possibility of erosion of the bluff face as discussed in the "Bluff Erosion" section of this report.

RECOMMENDATIONS

EARTHWORK AND GRADING

GENERAL: All grading should conform to the guidelines presented in Appendix Chapter A33 of the Uniform Building Code, the minimum requirements of the City of Encinitas, and the recommended Grading Specifications and Special Provisions attached hereto, except where specifically superseded in the text of this report. Prior to grading, a representative of Christian Wheeler Engineering should be present at the pre-construction meeting to provide additional grading guidelines, if necessary, and to review the earthwork schedule.

OBSERVATION OF GRADING: Continuous observation by the Geotechnical Consultant is essential during the grading operation to confirm conditions anticipated by our investigation, to allow adjustments in design criteria to reflect actual field conditions exposed, and to determine that the grading proceeds in general accordance with the recommendations contained herein.

CLEARING AND GRUBBING: Site preparation should begin with the demolition and removal of the existing improvements at the site that are designated for removal. This removal should include all existing foundations, slabs, pavements, and above grade and underground utilities, as well as any vegetation, trees, and other deleterious materials, including all root balls from trees and all significant root material. The resulting organic materials and construction debris should be disposed of in an appropriate off-site facility.

SITE PREPARATION: After clearing and grubbing, site preparation should consist of the removal of any and all weathered terrace deposits, natural topsoils, undocumented fill materials and soil disturbed during the removal of the existing improvements from areas to support fill and/or settlement-sensitive improvements. These materials should be removed to the contact with competent terrace deposits. The thickness of the unsuitable and disturbed materials is expected to range from about one foot to five feet below the existing grades. We expect that planned site grading, which will include cuts of up to about ten feet from existing site grades to create the proposed subterranean level of the residence, will remove most if not all of these materials from the area of the proposed residence. We anticipate that the proposed residence will have setbacks of approximately 10 feet from the northern and southern property lines. As such, the removals should extend at least five feet outside the perimeter of the residence and two feet outside areas to support settlement sensitive exterior improvements, such as the driveway walkways and patios. Where necessary to achieve planned site grades, the excavated materials should mixed and moisture conditioned and replaced as compacted structural fill. The bottom of the excavations should be approved by the Geotechnical Consultant prior to placing fills or constructing improvements.

PROCESSING OF FILL AREAS: Prior to placing any new fill soils or constructing any new improvements in areas that have been cleaned out to receive fill and approved by the geotechnical consultant or his representative, the exposed soils should be scarified to a depth of 12 inches, moisture conditioned, and compacted to at least 90 percent relative compaction.

COMPACTION AND METHOD OF FILLING: All structural fill placed at the site should be compacted to a relative compaction of at least 90 percent of maximum dry density as determined by ASTM Laboratory Test D1557. Fills should be placed at or slightly above optimum moisture content, in lifts six to eight inches thick, with each lift compacted by mechanical means. Fills should consist of approved earth material, free of trash or debris, roots, vegetation, or other materials determined to be unsuitable by our soil technicians or project geologist. Fill material should be free of rocks or lumps of soil in excess of six inches in maximum dimension. Based on our subsurface observations and laboratory testing, we anticipate the on-site soils will be suitable for use as structural fill. All utility trenches should be compacted to a minimum of 90 percent of its maximum dry density.

SURFACE DRAINAGE: Pad drainage should be designed to collect and direct surface water away from the proposed structure and the bluff top and toward approved drainage areas. For earth areas, a minimum gradient of one percent should be maintained. The ground around the proposed building and near the edge of the bluff should be graded so that surface water flows rapidly away from the building and bluff top without ponding. In general, we recommend that the ground adjacent to buildings slope away at a gradient of at least two percent. Densely vegetated areas where runoff can be impaired should have a minimum gradient of five percent within the first five feet from the structure. Where possible, drainage should be directed to suitable disposal areas via non-erodible devices such as paved swales, gunited brow ditches, and storm drains. Eave gutters and downspouts should discharge into controlled drainage devices.

Furthermore, as presented in Section 30.34.020 B of the City's Municipal Code (as amended by Ordinance 95-04), the homeowner and contractor should realize that "Irrigation shall be limited to hose bibs or water saving irrigation systems with automatic timers. No permanent irrigation systems shall be permitted within 40 feet of the coastal bluff edge." Section 30.34.020 B of the City's Municipal Code further states that the use of ice plant should be avoided and that native and drought-tolerant plant life should be emphasized. Such provisions in the Municipal Code are intended to minimize irrigation requirements and to reduce the potential for slide hazards due to over-watering. Furthermore, we recommend that temporary irrigation systems should have shut off and control valves located to the east of the recommended 40-foot coastal bluff top setback area.

FOUNDATIONS

GENERAL: The proposed residence may be supported by conventional shallow foundations or a deep foundation system consisting of augured, cast-in-place concrete piers that are tied together with grade beams. End bearing within the competent portions of the terrace deposits should be used for pier support. Improvements associated with, but not part of, the residence may be supported by conventional shallow foundations.

The shear pins, which are proposed along the edge of the required bluff top setback may or may not serve a double function of providing vertical bearing support for the proposed structure. If the proposed shear pins are utilized to provide bearing support for the proposed residence or any other settlement-sensitive structure, in no case should the depths of the shear pins be lessened from the required shear pin depth presented in the following section of this report.

CONVENTIONAL FOUNDATIONS

FOOTING DIMENSIONS: New continuous and spread footings supporting the proposed residence should be embedded at least 18 and 24 inches below finish pad grade for two- and three-story portions of the structure, respectively. Continuous and isolated footings should have a minimum width of 12 inches and 24 inches, respectively.

BEARING CAPACITY: New continuous and spread footings with the above minimum dimensions may be designed for an allowable soil bearing pressure of 2,500 pounds per square foot. This value may be increased by one-third for combinations of temporary loads such as those due to wind or seismic loads.

FOOTING REINFORCEMENT: A structural engineer should provide reinforcement requirements for foundations. However, based on the existing soil conditions, we recommend that the minimum reinforcing for continuous footings supporting any lateral additions consist of at least two No. 5 bars positioned three inches above the bottom of the footing and two No. 5 bars positioned approximately two inches below the top of the footing.

LATERAL LOAD RESISTANCE: Lateral loads against foundations may be resisted by friction between the bottom of the footing and the supporting soil, and by the passive pressure against the footing. The coefficient of friction between concrete and soil may be considered to be 0.35. The passive resistance may be considered to be equal to an equivalent fluid weight of 350 pounds per cubic foot. This assumes the footings are poured tight against undisturbed soil. If a combination of the passive pressure and friction is used, the friction value should be reduced by one-third.

SETTLEMENT CHARACTERISTICS: The anticipated total and/or differential settlement is expected to be less than about one-half inch for new foundations, provided the recommendations presented in this report are followed. It should be recognized that minor cracks normally occur in concrete slabs and foundations due to concrete shrinkage during curing or redistribution of stresses, therefore some cracks should be anticipated. Such cracks are not necessarily an indication of excessive vertical movements.

EXPANSIVE CHARACTERISTICS: The anticipated foundation soils are expected to have a low expansion potential (E.I. less than 50). The recommendations presented in this report reflect this condition.

FOUNDATION PLAN REVIEW: The foundation plans should be submitted to this office for review in order to ascertain that the recommendations of this report have been implemented, and that no additional recommendations are needed due to changes in the anticipated construction.

FOUNDATION EXCAVATION OBSERVATION: All foundation excavations should be observed by the Geotechnical Consultant prior to constructing forms or placing reinforcing steel to determine if the foundation recommendations presented herein are complied with. All footing excavations should be excavated neat, level and square. All loose or unsuitable material should be removed prior to the placement of concrete.

DEEP FOUNDATION SYSTEMS

MINIMUM PIER DIMENSIONS: Drilled, cast-in-place concrete piers may be used to support the proposed structure. We recommend that piers be embedded at least 10 feet below finish pad grade. At this depth, we anticipate that the piers will be supported by the competent portions of the terrace deposits that underlie the site. Piers should have a minimum diameter of 24 inches and, as a minimum, should be spaced no closer than three pier diameters. The project structural engineer should design all pier locations, dimensions, and pier reinforcing using the recommendations and design parameters herein.

PIER REINFORCING: Piers should be reinforced in accordance with the recommendations of the project structural engineer. The reinforcing cages should extend the full depth of the piers.

BEARING CAPACITY: Incorporating the minimum dimensions recommended, cast-in-place concrete piers may be designed for an allowable downward axial bearing capacity of 10 kips per square foot. This value may be increased by 800 psf for each additional foot of pier embedment over 10 feet, up to a maximum allowable bearing capacity of 30 kips per square foot.

LATERAL PIER CAPACITY: The passive pressure for the lower terrace deposits may be considered to be 350 pounds per square foot per foot of depth, up to a maximum value of 3,500 psf. This value may be assumed to act on an area equal to twice the pier diameter.

CLEANING OF PIER EXCAVATIONS: Loose or unsuitable material should be removed from the foundation excavations prior to the placement of concrete. Cleaning of the bottom of the pier excavations may be performed by careful operations of the driller and back-spinning the drill auger under pressure or utilizing a clean-out plate. Hand cleaning may also be required.

SHEAR PINS

GENERAL: In order to determine the necessary parameters for the design of a shear pin system to increase the gross stability of the site such that the site will demonstrate minimum factors-of-safety of 1.5 and 1.1 or greater against static and pseudo-static slope failures, respectively, at and east of the recommended bluff top setback (40 feet) we have performed a series of quantitative bluff stability analyses to model the proposed site conditions described herein.

As indicated by the results of our slope stability analyses presented in Appendices B through D of this report, we have determined that a single row of shear pins, which if installed 40-feet back from the existing top of bluff, will increase the minimum factors-of-safety against static and pseudo-static slope failures to 1.5 and 1.1, respectively, provided the shear pins are designed to resist a lateral force of 35,000 pounds for each linear foot of slope. If necessary, overlapping and adjacent rows of shear pins may be installed to achieve a force of 35,000 pounds for each linear foot of slope. It should be recognized that the 40-foot setback distance from the existing top of bluff to the row of shear pins described above, was modeled in our analyses to represent the currently proposed site development concept described to us.

The shear pins should consist of drilled, cast-in-place, reinforced concrete piers. The shear pins should be anchored into the underlying, terrace deposits and be minimally dimensioned in accordance with the recommendations presented below. .

LATERAL LOADS ON SHEAR PINS: As described above, the shear pins should be designed to resist a force of 35,000 pounds for each linear foot of slope. For example, for ten-foot, center-to-center spacing of the shear pins, each shear pin will need to be designed to resist of lateral load of 350,000 pounds. In general, this load may be assumed to act above the base of the critical failure surface a distance of one-third the height of the pier that is above the base of the critical failure surface. Specific point of application will need to be determined once the site layout and shear pin locations are finalized

MINIMUM SHEAR PIN DIMENSIONS: The recommended shear pins should be founded within dense to very dense terrace deposits. As a minimum, the shear pins should be embedded at least 40 feet below existing site grades; however, the depth may be greater to satisfy the required lateral capacities of the proposed shear pins. Shear pins should have a minimum diameter of 24 inches. The project structural engineer should design all shear pin locations, dimensions, and pier reinforcing using the recommendations and design parameters herein. However, the shear pins should be spaced no farther than three pier diameters.

SHEAR PIN REINFORCING: Piers should be reinforced in accordance with the recommendations of the project structural engineer. The reinforcing cages should extend the full depth of the piers.

SHEAR PIN LATERAL CAPACITY: The passive pressure for the competent terrace deposits below the base of the critical failure surface may be considered to be 450 pounds per square foot per foot of depth, up to a maximum value of 6,500 psf. This value may be assumed to act on an area equal to twice the pier diameter. As described above, the specific point of application will need to be determined once project plans are finalized. The existing terrace deposits above the base of the critical failure surface should not be included in passive pressure calculations. Similar to shoring applications, the shear pins should be designed as cantilevered members such that they experience no more than ¼-inch of deflection at their tops.

CLEANING OF SHEAR PIN EXCAVATIONS: Loose or unsuitable material should be removed from the shear pin excavations prior to the placement of concrete. Cleaning of the bottom of the excavations may be performed by careful operations of the driller and back-spinning the drill auger under pressure or utilizing a clean-out plate.

SHEAR PIN EXCAVATION OBSERVATION: All pier excavations should be observed by the Geotechnical Consultant prior to placing the reinforcing steel cage to determine if the soil and geologic conditions are similar to the conditions anticipated in the preparation of this report.

SHEAR PIN PLAN REVIEW: It is recommended that the shear pin plans be submitted to this office for review in order to verify that the recommendations presented in this report are incorporated in the structural plans.

ON-GRADE SLABS

GENERAL: It is our understanding that the floor system of the proposed residence will consist of a concrete slab-on-grade. The following recommendations are considered the minimum slab requirements based on the soil conditions and are not intended in lieu of structural considerations. All slabs should be designed by a qualified structural engineer.

INTERIOR FLOOR SLABS: The minimum floor slab thickness should be four inches (actual) and all floor slabs should be reinforced with at least No. 3 reinforcing bars placed at 18 inches on center each way. Slab reinforcement should be supported on chairs such that the reinforcing bars are positioned at mid-height in the floor slab. The slab reinforcement should extend into the perimeter foundations at least six inches.

MOISTURE PROTECTION FOR INTERIOR SLABS: For interior on-grade floor slabs of the residence that may be covered with moisture-sensitive flooring, steps should be taken to minimize the transmission of moisture vapor from the subsoil through the slabs where it can potentially damage the interior floor coverings. Local industry standards typically include the placement of a vapor retarder, such as plastic, in a layer of coarse sand placed directly beneath the concrete slab. Two inches of sand and four inches of sand are typically used under and below the plastic, respectively. This is the most common under-slab vapor retarder system used in San Diego County. The vapor retarder should be at least 15-mil plastic with sealed seams and should extend at least 12 inches down the sides of the interior and perimeter footings. The sand should have a sand equivalent of at least 30, and contain less than 20% passing the Number 100 sieve and less than 10% passing the Number 200 sieve. It is recommended that the basement slab be underlain by a six-inch-thick layer of gravel in lieu of the lower sand layer. The gravel should be wrapped in filter fabric such as Mirafi 140N. Additional steps may be taken to minimize moisture vapor transmission through the basement slabs. These may include changes or additions to typical concrete mixtures or waterproofing applications. Specific recommendations can be provided by this office upon request.

Although the system described above has historically performed adequately, national standards for the installation of vapor retarders below interior slabs are changing as evidenced in currently published standards including ACI 302, "Guide to Concrete Floor and Slab Construction" and ASTM E1643, "Standard Practice for Installation of Water Vapor Retarder Used in Contact with Earth or Granular Fill Under Concrete Slabs". Rather than placing the vapor retarder between the two sand layers, both of these standards recommend placing the sand capillary break layer onto the subgrade with a vapor retarder placed above the sand and the concrete placed directly onto the vapor retarder. There are advantages and disadvantages to each of these installation procedures.

An advantage to placing concrete directly onto a vapor retarder is that it eliminates the layer of sand between the slab and vapor retarder. This layer of sand typically contains moisture prior to the placement of concrete and can receive more moisture during the curing and construction processes. This moisture can be retained in the sand layer for an extended period of time until the concrete moisture decreases to the point at which the excess sand moisture is absorbed by the concrete and transmitted up through the slab. This process can take many months depending upon the environmental conditions.

One disadvantage to placing concrete directly onto a vapor retarder is that removing the sand layer from directly beneath the concrete restricts the ability of the concrete to lose moisture on both the top and bottom surfaces during the initial curing period. Variations in the drying rate between the top and bottom surfaces can result in increased concrete cracking, curling, and other finishing issues. The drying rate differences and their potential side effects can be minimized, however, with suitable finishing and curing procedures.

Recognizing the stated benefits and limitations of these standard below-slab vapor retarder systems, the owner and designer should select the system that they believe is most suitable for this project considering the construction schedule and planned floor coverings. It should be understood that neither of the described systems provides a "waterproof barrier". It should also be understood that slab concrete contains free water and should be allowed to reach equilibrium in an environment similar to that anticipated in the completed structure prior to installing floor coverings. We recommend that the flooring installer perform standard moisture vapor emission tests prior to the installation of all moisture-sensitive floor coverings in accordance with ASTM F1869 "Standard Test Method for Measuring Moisture Vapor Emission Rate of Concrete Subfloor Using Anhydrous Calcium Chloride".

EXTERIOR CONCRETE FLATWORK: Exterior concrete slabs on grade should have a minimum thickness of four inches and should be reinforced with at least No. 3 bars placed at 18 inches on center each way (ocew). Where walkway or patio slabs abut perimeter foundations, they should be doweled into the footings. Driveway slabs should have a minimum thickness of 5 inches and be reinforced with at least No. 4 bars placed at 18 inches ocew. Driveway slabs should be provided with a 12-inch-thick edge. Concrete sections for stamped concrete should be measured below the stamped depth.

It should be expected that lightweight exterior improvements such as concrete flatwork and curb and gutters that are underlain by expansive soils can experience some amount of heave damage even if thickened and more heavily reinforced. The potential for heave damage to exterior improvements can be lessened by placing a two-foot-thick mat of sandy, non-detrimentally expansive soils with an expansion index less than 50 below the improvements; however, the decision to do so is an economic decision that will need to be made by the owner.

A concrete mix with a 1-inch maximum aggregate size and a water/cement ratio of less than 0.6 is recommended for exterior slabs. Lower water content will decrease the potential for shrinkage cracks. Consideration should be given to using a concrete mix for the driveway that has a minimum compressive strength of 3,000 pounds per square inch. This suggestion is meant to address early driveway use prior to full concrete curing. Both coarse and fine aggregate should conform to the latest edition of the "Standard Specifications for Public Works Construction" ("Greenbook").