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DRAFT ENVIRONMENTAL IMPACT REPORT
ALAMITOS BAY MARINA REHABILITATION PROJECT
CITY OF LONG BEACH

APPENDIX E
GEOTECHNICAL EVALUATION

**GEOTECHNICAL EVALUATION
ALAMITOS BAY MARINA IMPROVEMENT
MARINA DRIVE
LONG BEACH, CALIFORNIA**

PREPARED FOR:

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February 9, 2007
Project No. 206918001

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Mr. Ken Johnson
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Subject: Geotechnical Evaluation
Alamitos Bay Marina Improvement
Marina Drive
Long Beach, California

Dear Mr. Johnson:

In accordance with your request and authorization, we have performed a geotechnical evaluation for the proposed improvements to the Alamitos Bay Marina in Long Beach, California. Our evaluation was performed to assess the soil and geologic conditions at the project site and to develop geotechnical recommendations for the design and construction of the improvements. This report presents our geotechnical findings, conclusions, and recommendations relative to the project.

We appreciate the opportunity to be of service on this project.

Respectfully submitted,
NINYO & MOORE

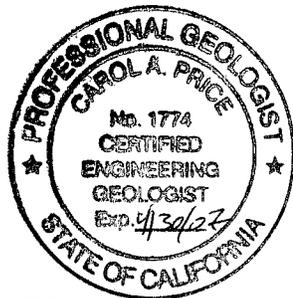

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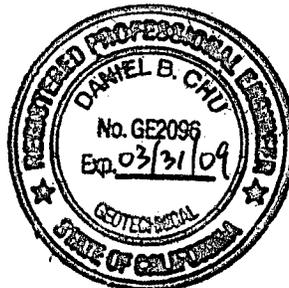


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1. INTRODUCTION

In accordance with your request and authorization, we have performed a geotechnical evaluation for the proposed improvements to the Alamitos Bay Marina located in Long Beach, California. We understand that the proposed improvements will include the replacement of boat slips in Basins 1 through 7, a new crane platform in Basin 4, new restroom facilities, and new roadways and parking areas. The purpose of our geotechnical evaluation was to assess the soil and geologic conditions at the project site and to develop recommendations for the geotechnical aspects of design and construction of the proposed improvements. This report presents our geotechnical findings, conclusions, and recommendations regarding the proposed improvements.

2. SCOPE OF SERVICES

Our scope of services included the following:

- Project coordination and review of readily available background materials pertaining to the project, including geologic maps and literature, stereoscopic aerial photographs, in-house information, and as-built plans for the existing boat slip guide piles.
- Geotechnical site reconnaissance to observe the general site conditions, to select and mark our proposed boring and cone penetrometer test (CPT) locations, and to coordinate with Underground Service Alert for utility clearance.
- Subsurface exploration consisting of seven CPTs to depths of approximately 51 to 83½ feet below the existing ground surface. In addition, our subsurface exploration included the drilling, logging, and sampling of 13 small-diameter exploratory borings. Seven borings were drilled onshore to depths of approximately 20½ to 61½ feet below the existing ground surface, and six of the borings were drilled offshore to depths of approximately 51½ to 66½ feet below the existing ocean floor. The borings were logged by a representative of our firm, and bulk and relatively undisturbed soil samples were collected at selected intervals for laboratory testing.
- Laboratory testing of selected soil samples, including evaluation of in-situ moisture content and dry density, sieve analysis, percent of materials finer than the No. 200 sieve, Atterberg limits, shear strength, triaxial unconsolidated undrained compression, R-value, sand equivalent, and soil corrosivity.
- Data compilation and geotechnical analyses of the information obtained from our background review, surface evaluation, and laboratory testing.

- Preparation of this report presenting our findings, conclusions, and recommendations for the proposed project.

3. SITE DESCRIPTION

The Alamitos Bay Marina is located at the intersection of State Highway 1 (Pacific Coast Highway) and Second Street, within Los Angeles County in Long Beach, California (Figure 1). The site is irregular in shape and is bounded by Pacific Coast Highway to the northeast, the San Gabriel River to the southeast, Ocean Boulevard to the southwest, and Bay Shore Avenue and the Los Cerritos Channel to the northwest.

Alamitos Bay is a body of water located adjacent to the Pacific Ocean and contains both Naples Island and the Long Beach peninsula. The site is situated on relatively level terrain at an elevation between approximately 5 and 10 feet above mean sea level (MSL) (United States Geological Survey [USGS], 1981).

Alamitos Bay is a 50+ year old marina with approximately 1991 boat slips located throughout the Alamitos Bay area as stated on the Long Beach Marine Bureau (parks, recreation, and marine development) website. There are seven basins within Alamitos Bay. Basins 1 through 5 are located near the easterly portion of the bay and are primarily used as permanent boat slips basins. Basins 6 and 7 are located at the northwest and south of the bay, respectively, and are used primarily as waterways around Naples Island and along the Long Beach peninsula. The marina is primarily surrounded by residential properties, with commercial properties located along Second Street and Pacific Coast Highway. A majority of the marina perimeter is lined with parking areas, with a few structures, including restrooms, yacht clubs, restaurants, and other miscellaneous structures.

4. SITE BACKGROUND

The Alamitos Bay Marina, as it exists today, was formed as a result of a series of dredging activities that began in the early 1900s. In the 1950s, portions of the bay were dredged to form a circular waterway and the existing basins and boat slips were constructed. During this develop-

ment, the existing guide piles along the boat slips were placed. Based on our review of as-built plans provided by the City of Long Beach, the existing guide piles are 14-inch-diameter, concrete jacketed and capped, Douglas-fir timber piles and are 40 feet in length.

5. PROPOSED CONSTRUCTION

We understand that the proposed construction consists of offshore and onshore improvements. The waterside improvements will include the removal and replacement of the existing boat slips, including the guide piles, within Basins 1 through 7. The purpose of these improvements is to increase the size of the existing boat slips to accommodate larger boats. The landside improvements will include construction of a new crane platform within Basin 4 and new restroom facilities, roadways and parking areas, and miscellaneous improvements throughout the marina.

6. SUBSURFACE EVALUATION AND LABORATORY TESTING

Our onshore subsurface exploration at the subject site was performed on November 20, 21, and 22, 2006, and consisted of seven CPTs and the drilling, logging, and sampling of seven small-diameter borings. The approximate locations of the CPT soundings and the exploratory borings are shown on Figure 2. The CPTs ranged in depth from approximately 51 to 83½ feet below the existing ground surface. The borings were drilled with a truck-mounted drill rig utilizing 8-inch-diameter, hollow-stem augers. The borings were drilled to depths of approximately 20½ feet to 61½ feet below the existing ground surface and were logged by a representative from our firm. Bulk and relatively undisturbed soil samples were obtained at selected depths for laboratory testing. The logs of the exploratory borings are presented in Appendix A, and the CPT sounding logs are presented in Appendix B.

Our offshore subsurface evaluation at the subject site was performed between January 4 and 11, 2007, and consisted of the drilling, logging, and sampling of six small-diameter borings. The approximate locations of the exploratory borings are shown on Figure 2. The borings were drilled with a barge-mounted drill rig utilizing 8-inch-diameter, hollow-stem casing. The borings were drilled to depths of approximately 50½ feet to 66½ feet below the existing ground surface and

were logged by a representative from our firm. Bulk and relatively undisturbed soil samples were obtained at selected depths for laboratory testing. The logs of the exploratory borings are presented in Appendix A.

Laboratory testing of representative soil samples was performed to evaluate in-situ moisture content and dry density, gradation, percent of materials finer than the No. 200 sieve, Atterberg limits, shear strength, triaxial unconsolidated undrained compression, R-value, sand equivalent, and soil corrosivity. The results of our in-situ moisture content and dry density evaluation are presented on the boring logs in Appendix A. The remaining laboratory testing results are presented in Appendix C.

7. GEOLOGY AND SUBSURFACE CONDITIONS

7.1. Regional Geology

The subject site is located in the southwestern block of the Los Angeles Basin in the coastal plain of the Peninsular Ranges Geomorphic Province (Norris and Webb, 1990). The geomorphic province encompasses an area that extends approximately 125 miles from the Transverse Ranges and the Los Angeles Basin south to the Mexican border and the tip of Baja California (Norris and Webb, 1990). The Peninsular Ranges vary in width from approximately 30 to 100 miles and are generally characterized by northwest trending mountain ranges separated by subparallel fault zones. The major structural fault systems bounding this area include the southern onshore segment of the Newport-Inglewood fault located approximately 0.6 mile northwest of the site, as well as the potentially active Los Alamitos and Norwalk fault zones located approximately 3 miles northeast and 10.8 miles northeast, respectively (Jennings, 1994). In general, the mountain ranges are underlain by Jurassic-age metavolcanic and metasedimentary rocks and Cretaceous-age igneous rocks of the southern California batholith. Regional geologic mapping indicates that the project area is underlain by Holocene-age stream channel, alluvial fan, and flood plain deposits consisting of clay, silt, sand, and cobbles.

7.2. Site Geology

The Alamitos Bay Marina lies to the northwest of the San Gabriel River and south of the outlet of the Los Cerritos Channel. The San Gabriel River borders the southeastern perimeter of the marina and trends in a northeast-southwest direction. The Los Cerritos Channel roughly trends in a northeast-southwest direction and joins the Marina at its northern tip. Published geologic maps and literature indicate that the site is underlain by artificial fill consisting of sand and silty sand (California Division of Mines and Geology [CDMG], 1998). According to the State of California Seismic Hazard Zone maps of the project site, on-site soils are mapped as potentially susceptible to earthquake-induced liquefaction (Figure 3) (CDMG, 1998 and 1999).

During our subsurface exploration, we encountered fill materials to depths ranging from approximately 7 to 15½ in our onshore borings. The fill materials generally consisted of medium dense, clayey sand, medium dense, silty sand, and very stiff, sandy clay with trace gravel, wood shards, and shells. Alluvial deposits were encountered beneath the fill to the explored depth of approximately 83½ feet. The alluvial deposits generally consisted of inter-layered sand, clay, and silt. The sand material encountered generally ranged from loose to very dense, silty sand and medium dense, poorly graded sand to silty sand. The clay material encountered generally ranged from very soft to hard, silty clay, and sandy clay. The silt generally ranged from very loose to dense, sandy silt and firm to hard, clayey silt. More detailed descriptions of the subsurface materials are presented on the boring logs in Appendix A.

7.3. Groundwater

Due to the subject site's close proximity to the Pacific Ocean, bentonite slurry was used in the majority of our onshore borings, and, therefore, groundwater depths were difficult to evaluate. In our Boring B-1, groundwater was measured at an approximate depth of 35 feet below the ground surface; however, due to the fine-grained nature of the soils encountered in the boring and the limited time for which the boring was left open, groundwater is anticipated to be much shallower. According to the CDMG (1998), historic high groundwater is estimated to be approximately 8 feet below the existing ground surface. However, fluctua-

tions in groundwater levels may occur due to tidal fluctuations, variations in precipitation, ground surface topography, subsurface stratification, irrigation, and other factors which may not have been evident at the time of our field evaluation.

8. FAULTING AND SEISMICITY

Based on our review of referenced geologic maps and stereoscopic aerial photographs, the ground surface in the vicinity of the subject site is not mapped as being transected by any known active or potentially active fault; therefore, the potential for surface fault rupture is considered to be low. Additionally, the site is not located within a State of California Earthquake Fault Zone (Alquist-Priolo Special Studies Zone, Hart and Bryant, 1997). However, the subject site is located in a seismically active area, as is the majority of southern California, and the potential for strong ground motion at the site is considered significant. Figure 4 shows the approximate site location relative to the major faults in the region. The nearest known active fault is the Newport-Inglewood fault, located approximately 0.6 mile northwest of the project site.

Table 1 lists selected principal known active faults that may affect the subject site, the maximum moment magnitude (M_{max}) as published by the California Geological Survey (CGS) by Cao, et al. (2003), and the type of fault as defined in Table 16-U of the California Building Code (CBC, 2001). The approximate fault to site distances were calculated by the computer program FRISKSP developed by Blake (2001a).

Table 1 – Principal Active Faults

Fault	Approximate Fault to Site Distance in miles (km)	Maximum Moment Magnitude ¹ (M_{max})	Fault Type ²
Newport-Inglewood (Los Angeles Basin)	0.6 (0.9)	7.1	B
Palos Verdes	7.9 (12.7)	7.3	B
San Joaquin Hills (Blind Thrust)	10.9 (17.5)	6.6	B
Puente Hills (Blind Thrust)	12.9 (20.8)	7.1	B
Whittier	16.9 (27.2)	6.8	A
Upper Elysian Park (Blind Thrust)	21.4 (34.4)	6.4	B
San Jose	23.9 (38.4)	6.4	B
Raymond	25.4 (40.8)	6.5	B
Hollywood	26.3 (42.3)	6.4	B
Verdugo	26.3 (42.4)	6.9	B
San Andreas – 1857 Rupture	49.5 (79.6)	7.8	A
Notes:			
¹ Cao et al., 2003.			
² CBC, 2001; Cao et al., 2003.			

The principal seismic hazards at the subject site are surface ground rupture, ground shaking, seismically induced liquefaction, and various manifestations of liquefaction-related hazards (e.g., dynamic settlement and lateral spread). A brief description of these hazards and the potential for their occurrences on site are discussed below.

8.1. Ground Rupture

The probability of damage from surface ground rupture is low due to the lack of known active faults directly underlying the subject site. Surface ground cracking related to shaking from distant events is not considered a significant hazard, although it is a possibility.

8.2. Ground Motion

Our evaluation of the ground shaking hazard included review of a probabilistic seismic hazard assessment that consisted of statewide estimates of peak horizontal ground accelerations conducted for California (Peterson, et al., 1996). In addition, for the purposes of evaluating seismically induced geotechnical hazards at the site, a site-specific probabilistic seismic hazard analysis was performed to evaluate anticipated peak ground accelerations (PGAs) us-

ing the computer program FRISKSP developed by Blake (2001c). A probabilistic analysis incorporates uncertainties in time, recurrence intervals, size, and location (along faults) of hypothetical earthquakes. This method thus accounts for likelihood (rather than certainty) of occurrence and provides levels of ground acceleration that might be more reasonably hypothesized for a finite exposure period. FRISKSP calculates the probability of occurrence of various ground accelerations at a site over a period of time and the probability of exceeding expected ground accelerations within the lifetime of the proposed structures from the significant earthquakes within a specific radius of search. For the present case, a search radius of 62 miles (100 kilometers) was selected. The earthquake magnitudes used in this program are based on the current CGS fault model.

The published guidelines of CGS (2004) define a PGA with a 10 percent probability of exceedance in 50 years as the Design Basis Earthquake (PGA_{DBE}) ground motion, and this value is typically used for residential, commercial, and industrial structures. The PGA with a 10 percent probability of exceedance in 100 years is defined as the Upper Bound Earthquake (PGA_{UBE}) ground motion and is used for public schools, hospitals, and other essential facilities in California. The statistical return periods for the PGA_{DBE} and PGA_{UBE} are approximately 475 and 949 years, respectively.

In evaluating the seismic hazards associated with the subject site, we have considered a PGA that has a 10 percent probability of being exceeded in 50 years (i.e., PGA_{DBE}) and used an attenuation relation proposed by Boore, et al. (1997), for soil (with an average shear wave velocity of 310 meters per second). The PGA_{DBE} for the site was 0.34g when weighted to an earthquake magnitude of 7.5. The PGA_{DBE} increases to 0.40g when no magnitude weighting factor is considered in probabilistic seismic hazard analysis. These estimates of ground motion do not include near-source factors that may be applicable in the design of structures on site.

8.3. Liquefaction

Liquefaction is the phenomenon in which loosely deposited, saturated, granular soils (located below the water table) with clay contents (particles less than 0.005 millimeters [mm]) of less than 15 percent, liquid limit of less than 35 percent, and natural moisture content greater than 90 percent of the liquid limit undergo rapid loss of shear strength due to development of excess pore pressure during strong earthquake-induced ground shaking. Ground shaking of sufficient duration results in the loss of grain-to-grain contact due to a rapid rise in pore water pressure, and it eventually causes the soil to behave as a fluid for a short period of time. Liquefaction is known generally to occur in saturated or near-saturated cohesionless soils at depths shallower than 50 feet below the ground surface. Factors known to influence liquefaction potential include composition and thickness of soil layers, grain size, relative density, groundwater level, degree of saturation, and both intensity and duration of ground shaking.

The liquefaction potential at the project site was evaluated using the CPT sounding results, the soil sampler blow counts recorded from the exploratory borings, and our laboratory test results. In addition, the historic high groundwater of 8 feet below the ground surface was considered for our liquefaction evaluation. The liquefaction analysis was based on the NCEER procedure (Youd, et al., 2001). For the analyses using the soil sampler blow counts, the computer program LIQUEFY2 (Blake, 2001a) was used to calculate the factor of safety against liquefaction. A magnitude-weighted PGA_{DBE} of 0.34g was used in our analysis for an earthquake magnitude of 7.5. Due to the variability of the on-site soils, the potential for liquefaction varies across the site. Our liquefaction analysis indicates that some of the granular soil layers located below the historic high groundwater level may liquefy during the design seismic event up to depths of approximately 48 feet below the ground surface for the on-shore portions of the site and to depths of approximately 14 feet below the ground surface in the offshore portions of the site.

8.4. Lateral Spreading

Lateral spreading of the ground surface during an earthquake usually takes place along weak shear zones that have formed within a liquefiable soil layer. Lateral spread has generally been observed to take place in the direction of a free-face (i.e., retaining wall, slope, channel) but has also been observed to a lesser extent on ground surfaces with gentle slopes. An empirical model developed by Youd, et al. (1995, revised 2001), is typically used to predict the amount of horizontal ground displacement within a site. For sites located in proximity to a free-face, the amount of lateral ground displacement is strongly correlated with the distance of the site from the free-face. Other factors, such as earthquake magnitude, distance from the earthquake epicenter, thickness of the liquefiable layers, and the fines content and particle sizes of the liquefiable layers, also affect the amount of lateral ground displacement. Based on our analysis, seismically induced lateral spread of approximately 1 to 11 feet is estimated to occur.

8.5. Dynamic Settlement of Saturated Soils

The phenomenon of soil liquefaction may result in several hazards including liquefaction-induced settlement. In order to estimate the amount of post-earthquake settlement, the method proposed by Tokimatsu and Seed (1987) is generally used in which the seismically induced cyclic stress ratios and corrected blow counts (N-values) are correlated to the volumetric strain of the soil. The amount of soil settlement during a strong seismic event depends on the thickness of the liquefiable layers and the density and/or consistency of the soils.

Based on our analysis, post-earthquake dynamic ground settlements ranging from approximately 5 to 27 inches are estimated to occur in relatively saturated soils located below the historic high groundwater to depths of up to approximately 48 feet.

9. CONCLUSIONS

Based on the results of our geotechnical evaluation, the proposed project is feasible from a geotechnical perspective. There are no known geotechnical conditions that would preclude the

construction of the proposed improvements provided the recommendations of this report and appropriate construction practices are followed. In general, the following conclusions were made based on our evaluation:

- Groundwater was measured at a depth of approximately 35 feet below the ground surface in boring B-1 at the time of our drilling; however, due to the fine-grained nature of the soils encountered in the boring and the limited length of time for which the hole was left open, groundwater is anticipated to be encountered at shallower depths. The historic high groundwater within the project site is estimated at approximately 8 feet below the existing ground surface. Therefore, groundwater should be anticipated and planned for by the contractor during construction of the onshore improvements.
- During our subsurface exploration, clayey, silty, and sandy soils were encountered. Although caving was not observed in our onshore borings at the time of drilling, near surface groundwater may cause caving of coarse-grained alluvial soils.
- We estimate a peak ground acceleration (PGA_{DBE}) of 0.34g for an earthquake magnitude of 7.5 at the subject site that has a 10 percent probability of being exceeded in 50 years. We also estimate a PGA_{DBE} of 0.40g when no magnitude weighting factor is applied.
- The subsurface soils below the historic high groundwater are susceptible to liquefaction during the design seismic event. Our analysis indicates that the depth of liquefaction varies across the site but reaches depths of up to approximately 48 feet during the anticipated seismic event.
- Seismically-induced lateral spread of approximately 1 to 11 feet is estimated to occur.
- Liquefaction-induced ground settlement is estimated to be in the range of approximately 5 to 27 inches in the upper approximately 48 feet.
- The site is not located within a State of California Earthquake Fault Zone (Alquist-Priolo Special Studies Zone). Based on our review of published geologic maps and aerial photographs, no known active or potentially active faults underlie the site. The potential for surface fault rupture at the site is considered to be low.

10. RECOMMENDATIONS

In the following sections, we provide geotechnical recommendations for the design and construction of the proposed improvements at the Alamitos Bay Marina. These recommendations are based on our evaluation of the site geotechnical conditions and our understanding of the planned

development, including anticipated loads. The proposed construction should also be performed in accordance with the requirements of applicable governing agencies.

10.1. Earthwork

The quantity and type of earthwork operations performed during construction will depend on the final design plans. It is our understanding that at the time of this report, the design plans are still in their preliminary stages, but it is anticipated that minor grading will be associated with the construction of new roadways, parking areas, access ramps, and other miscellaneous improvements. If, in the final design, the existing bathrooms, including their underground utilities and adjacent improvements, are removed, earthwork at the site will include infilling of depressions created by the removal of below-grade improvements and grading associated with the installation of peripheral improvements, such as utilities and asphalt concrete pavement. Earthwork recommendations presented in the following sections are based upon the assumption that the finish grades at the site will not be changed significantly. Earthwork should be performed in accordance with the requirements of applicable agencies, and the recommendations presented herein.

10.1.1. Construction Plan Review and Pre-Construction Conference

We recommend that the grading and foundation plans be submitted to Ninyo & Moore for review to check for conformance to the recommendations provided in this report. We further recommend that a pre-construction conference be held in order to discuss the grading recommendations presented in this report. The owner and/or their representative, the governing agencies' representatives, the civil engineer, Ninyo & Moore, and the contractor should be in attendance to discuss the work plan, project schedule, and earthwork requirements.

10.1.2. Site Preparation

Prior to commencing earthwork operations, the existing improvements and deleterious materials at the site should be cleared. The existing improvements may include single-story bathroom structures and associated utilities, concrete flatwork, asphalt concrete

pavement, small planter areas, and other miscellaneous improvements. Debris from the clearing operations should be disposed of off-site. Resulting holes due to removal of obstructions that extend below grade, such as slabs or underground utilities, should be removed and filled with compacted fill.

10.1.3. Excavation Characteristics

We anticipate that excavation within the fill and alluvial materials present on site may be accomplished with grading equipment in good operating condition. Based on the results of our subsurface exploration, we expect that the subsurface soils encountered will be interlayered clay, silt, and sand. Although significant oversized materials were not encountered in our borings, oversized materials may be encountered in the alluvial deposits during the installation of deep foundations. Thus, the contractor should be prepared to take appropriate measures to address the presence of oversized materials.

In our opinion, temporary slopes in the fill or native soils should be stable at inclinations of 1½:1 (horizontal to vertical) or flatter soils above the water table. Some surficial sloughing may occur, and temporary slopes should be evaluated in the field by a competent person in accordance with Occupational Safety and Health Administration (OSHA) criteria. The on-site soils should be considered as Type C soils in accordance with OSHA.

Excavations close to or below the groundwater will encounter wet and loose or soft ground conditions. Wet soils may be subject to pumping under heavy equipment loads. Pumping soils will need to be stabilized prior to the placement of fill.

10.1.4. Fill Material

In general, the on-site earth materials should be suitable for reuse as fill. On-site and import fill soils should be free of trash, debris, roots, vegetation, or deleterious materials. Fill should generally be free of rocks or hard lumps of material greater than approximately 4 inches in diameter. Rocks or hard lumps larger than about 4 inches in diameter should be broken into smaller pieces or should be removed from the site. Im-

ported materials should consist of clean, granular material with a low expansion potential, corresponding to an expansion index of 50 or less as evaluated in accordance with Uniform Building Code (UBC) Standard 18-2 (International Conference of Building Officials [ICBO], 1997). Import material should be submitted to the project geotechnical consultant for review prior to importing to the site. The corrosion potential of proposed imported soils should also be evaluated if structures will be in contact with the imported soils. The contractor should be responsible for the uniformity of import material brought to the site.

10.1.5. Fill Placement and Compaction

Fill, structure backfill, and trench backfill associated with the proposed construction activities should be placed and compacted in accordance with project specifications and sound construction practices. Fill materials should be compacted to a relative compaction of 90 percent as evaluated by the American Society for Testing and Materials (ASTM) D 1557. Aggregate base materials beneath pavements should be compacted to a relative compaction of 95 percent. Fill materials should be moisture conditioned to slightly above the optimum laboratory moisture content. The lift thickness for fill soils will vary depending on the type of compaction equipment used but should generally be placed in horizontal lifts not exceeding 8 inches in loose thickness. Fill should be tested for specified compaction level by the geotechnical consultant.

10.2. Utilities

We understand that new utility lines associated with the construction of the bathroom structures may be constructed as part of the project. Trenches/excavations should be in conformance with OSHA guidelines for shoring and/or temporary slopes.

10.2.1. Trench Excavations

Temporary excavations above groundwater up to approximately 4 feet in depth should be generally stable. Excavations which expose friable, cohesionless sands, however, may be subject to caving. Excavations that seem unstable, expose cohesionless sands, or

are deeper than 4 feet, should be shored or the sides of the excavation laid back to slope inclinations of approximately 1½ to 1 (horizontal to vertical) for Type C soils per OSHA classification. We recommend that the contractor take appropriate measures to protect workers. OSHA requirements pertaining to worker safety should be observed. In the event that trenches are planned that extend below groundwater, dewatering or other special construction provisions may be appropriate.

10.2.2. Trench Bedding

We recommend that utility lines be supported on 6 or more inches of granular bedding material such as sand with a sand equivalent (SE) value of 30 or higher. Bedding material should be placed around the pipe and 12 inches or more above the top of the pipe in accordance with specifications of the recent edition of the “Greenbook” (Standard Specifications for Public Works). Our laboratory test results indicate that the on-site near-surface soils are not suitable as use for bedding around the pipe. Special care should be taken not to allow voids beneath the pipe. Bedding material and compaction requirements should be in accordance with the recommendations of this report, the project specifications, and applicable requirements of the appropriate governing agency.

10.2.3. Trench Backfill

Based on our subsurface evaluation, the on-site soils should be generally suitable for reuse as backfill provided they are free of organic material, clay lumps, debris, and rocks greater than approximately 4 inches in diameter. We recommend that trench backfill materials be in conformance with the “Greenbook” specifications for structure backfill. Fill should be moisture-conditioned to at or slightly above the laboratory optimum. Wet soils should be allowed to dry to a moisture content near the optimum prior to their placement as trench backfill. Trench backfill should be compacted to a relative compaction of 90 percent as evaluated by ASTM D 1557 in pavement areas where base materials are not utilized. The upper 12 inches of backfill should be compacted to a relative compaction of 95 percent as evaluated by ASTM D 1557. Special care should be exercised to avoid damaging the pipe during compaction of the backfill.

10.3. Seismic Design Considerations

Design of the proposed improvements should comply with design for structures located in Seismic Zone 4 in accordance with applicable jurisdictions and building codes and the standard practices of the Structural Engineers Association of California. A soil profile factor of S_D may be utilized in the UBC (ICBO, 1997) seismic design. Additional UBC seismic design parameters are provided in Table 2.

Table 2 – 1997 Uniform Building Code Seismic Recommendations

1997 UBC Seismic Design Factor	Value
Seismic Zone Factor, Z	0.4
Seismic Source Type*	B
Near Source Factor, N_a	1.3
Near Source Factor, N_v	1.6
Soil Profile Type	S_D
Seismic Coefficient, C_a	0.57
Seismic Coefficient, C_v	1.02
* Faults are designated as Type A, B or C, depending on maximum moment magnitude and slip rates (Table 16-U of UBC [ICBO, 1997]).	

10.4. Deep Pile Foundation

10.4.1. Crane Platform

Due to the presence of potentially liquefiable soil layers between approximately 16 feet and 40 feet below the existing ground surface in the vicinity of Basin 4 and the presence of loose to medium dense, sandy soils up to depths of approximately 50 feet, shallow foundations without ground modification improvements are not suitable for the proposed crane platform. However, due to the size of the project, ground improvements may not be cost effective. As a result, we recommend that a deep foundation system be utilized. For our preliminary analysis, we considered 14-inch-diameter, precast, driven, concrete piles. Our analysis has taken into account the effects of potential negative skin friction in the event of liquefaction-induced dynamic settlement. Due to the existence of sheet piling in front of the proposed crane platform, additional lateral loads which may be induced by the lateral spreading were not considered in the pile design.

10.4.1.1. Axial Capacity

We evaluated the load-carrying capacities by assuming the use of 14-inch-diameter, driven reinforced concrete piles, with an embedment of 45 feet. The axial capacity was analyzed using the computer program SPILE (Federal Highway Administration [FHWA], 1993), as well as the procedures developed by Eslami and Fellenius (1997), our laboratory shear test results, and boring and CPT information. Results of our analysis indicate that for a 45-foot-deep driven pile, a design load of 100 tons for a 14-inch-diameter pile can be used. These values are also summarized in Table 3 below. Uplift capacities have also been calculated and provided in Table 3. The axial capacities presented below may be increased by one-third when considering loads of short duration such as wind or seismic forces. The negative skin friction caused by a liquefaction-induced dynamic settlement was also considered in our capacity analysis and factored into the ultimate pile capacity values. A minimum pile spacing of three pile diameters on centers should be maintained.

Table 3 – Axial Load Capacities for Crane Platform

Pile Type	Concrete Driven Pile
Pile Size	14-inch circular
Pile Length (ft)	Minimum 45
Pile Tip Elevation (ft, MSL)*	-35±
Allowable Axial Capacity (tons)	100
Uplift Capacity (tons)	60
<p>Notes: ft – feet MSL – mean sea level *Elevation based on USGS Topographic Map (1981). Specified tip elevation should be checked prior to construction.</p>	

10.4.1.2. Lateral Capacity

Lateral capacities for the 45-foot-deep foundations were evaluated for the fixed-head conditions at a lateral deflection of ¼ inch by using the computer program

LPILE Version 4M developed by Ensoft, Inc. (1999). The results of our analysis, based on the assumptions indicated above, are summarized in Table 4.

Table 4 – Driven Pile Lateral Load Capacities for Crane Platform

Pile Type	14-inch Circular Concrete	
Pile Length (ft)	45	45
Pile Head Condition	Fixed	Free
(Deflection at the top of pile)	¼ inch	¼
Lateral Capacity (tons)	7	3
Maximum Positive Moment (tons-ft)	10	10½
Maximum Negative Moment (tons-ft)	28	½
Depth to Maximum Positive Moment (ft)	9	6
Depth to Maximum Negative Moment (ft)	0	15
Depth to Zero Deflection (ft)	13	10
Notes: ft – feet Depth is measured from top of driven pile.		

10.4.1.3. Lateral Loading and Settlement

For lateral loading, driven piles in a group may be considered to act individually when the center-to-center spacing is greater than 3B (where B is the diameter of the driven pile or drilled pier) in the direction of normal loading and greater than 6B in the direction of parallel loading. Lateral load reduction factors are provided in Table 5.

Table 5 – Lateral Load Reduction Factors

Center-to Center Spacing for In-Line Loading	Ratio of Lateral Resistance of Pile Group to Single Pile
6B	1.0
5B	0.9
4B	0.8
3B	0.7

Settlement for the pile foundation system with the design working axial loads is anticipated to be less than approximately ½ inch. Differential settlement between adjacent piles is anticipated to be less than approximately ¼ inch.

10.4.2. Boat Slip Guide Piles

For our preliminary analysis, we considered 14-inch-diameter, precast, driven, concrete piles. Our analysis has taken into account the effects of potential negative skin friction in the event of liquefaction-induced dynamic settlement.

10.4.2.1. Axial Capacity

We evaluated the load-carrying capacities by assuming the use of 14-inch-diameter, driven, reinforced, concrete piles, with an embedment of 40 feet. The axial capacity was analyzed using the computer program SPILE, our laboratory shear test results, and boring information. Results of our analysis indicate that for a 40-foot-deep driven pile, a design load of 25 tons for a 14-inch-diameter pile can be used. These values are also summarized in Table 6 below. Uplift capacities have also been calculated and provided in Table 6. The axial capacities presented below may be increased by one-third when considering loads of short duration such as wind or seismic forces. The negative skin friction caused by a liquefaction-induced dynamic settlement was also considered in our capacity analysis and factored into the ultimate pile capacity values. A minimum pile spacing of three pile diameters on centers should be maintained.

Table 6 – Axial Load Capacities for Boat Slip Guide Piles

Pile Type	Concrete Driven Pile
Pile Size	14-inch circular
Pile Length (ft)	Minimum 45
Pile Embedment (ft)	Minimum 30
Pile Tip Elevation (ft, MSL)*	-40±
Allowable Axial Capacity (tons)	80
Uplift Capacity (tons)	60
Notes: ft – feet MSL – Mean Sea Level *Elevation based on Hydrographic Survey provided by Transystems. Specified tip elevation should be checked prior to construction.	

10.4.2.2. Lateral Capacity

Lateral capacities for the 40-foot-deep foundations were evaluated for the free-head condition using the computer program LPILE Version 4M developed by Ensoft, Inc. (1999). The results of our analysis, based on the assumptions indicated above, are summarized in Table 7.

Table 7 – Driven Pile Lateral Load Capacities for Boat Slip Guide Piles

Pile Type	Concrete
Pile Length (ft)	50
Pile Embedment (ft)	30
Pile Size	14-inch circular
Free-Head (Deflection at the top of pile, inch)	4
Lateral Capacity (tons)	1
Maximum Positive Moment (tons-ft)	24
Maximum Negative Moment (tons-ft)	-1
Depth to Maximum Positive Moment (ft)	22
Depth to Maximum Negative Moment (ft)	36
Depth to Zero Deflection (ft)	28
Notes: ft – feet Depth is measured from top of driven pile.	

10.5. Spread Footings

Shallow foundations were considered for the proposed bathroom structures. These structures can be designed on shallow footings with the understanding that these structures may be severely damaged during the design earthquake event. We recommend that the shallow, continuous, spread footings be founded at a depth of 24 inches below the lowest adjacent finished grade, and should have a width of 12 inches. Preparation of subgrade for footings should include overexcavation of near-surface soils to a depth of 2 feet below the footing bottom. The overexcavation should extend a horizontal distance on either side equal to the depth of the overexcavation or 5 feet, whichever is greater, and backfilled with compacted fill as described in Sections 10.1.4 and 10.1.5. Footings located adjacent to utility trenches should have their bearing surfaces situated below an imaginary 1:1 (horizontal to vertical) plane projected upward from the bottom edge of the adjacent utility trench.

The spread footings, as described above and bearing on compacted fill, may be designed using an allowable bearing capacity of 2,000 pounds per square foot (psf). The bearing capacity may be increased by one-third when considering loads of short duration, such as wind or seismic forces. The spread footings should be reinforced in accordance with the recommendations of the project structural engineer.

Footings bearing in compacted fill soils may be designed using a coefficient of friction of 0.3, where the total frictional resistance equals the coefficient of friction times the dead load. Footings may be designed using a passive resistance value of 300 psf per foot of depth for a level ground condition up to a value of 3,000 psf per foot. The allowable lateral resistance can be taken as the sum of the frictional resistance and passive resistance provided the passive resistance does not exceed one-half of the total allowable resistance. The passive resistance (including the maximum value) may be increased by one-third when considering loads of short duration such as wind or seismic forces.

The anticipated settlement due to the bearing loads under the foundations is anticipated to be less than 1 inch.

10.6. Construction Considerations

Consideration should be given to how construction-related activities may affect the site as a whole. Driven piles have been designed to derive support from side friction. Deep foundations should be installed within specified limits of vertical and horizontal alignment and should not exceed a batter of two percent over the length. Further, the top of the driven piles should be within 3 inches of the surveyed location.

Historic high groundwater levels onshore have been estimated to be at a depth of approximately 8 feet below the surface. As a result, groundwater may be encountered in any site excavations. The contractor should be prepared to take appropriate measures to address the presence of groundwater in excavations, including dewatering.

10.7. Retaining Walls

Based on our understanding of the project, retaining structures are anticipated for the access ramps situated between the parking lot and the concrete walkway adjacent to the existing sea wall. In addition, if below-grade structures are considered for the proposed project, active and at-rest earth pressures (equivalent fluid pressures [EFP]) of 34 and 53 pounds per cubic foot should be used for yielding walls and restrained walls, supporting level backfill, respectively. In accordance with the CBC (ICBO, 2002), the dynamic lateral earth pressure should be utilized for a retaining wall higher than 12 feet when measuring from the bottom of the wall footing to the top of the wall. It is our understanding that the retaining walls constructed for this project will all be less than 12 feet in height, and, therefore, the dynamic lateral earth pressures for below-grade walls were not evaluated. If any retaining walls greater than 12 feet in height are constructed, the geotechnical engineer should provide the dynamic lateral earth pressures for design. The recommended lateral earth pressure values and considerations applicable for this project are summarized on Figures 5 and 6.

We recommend that if retaining walls are constructed as part of the site improvements, the retaining wall backfill should consist of select granular, free-draining material. Retaining wall backfill should conform with "Greenbook" recommendations for structural backfill.

Clayey fill and alluvial soils encountered in the borings and classified as CL are not suitable for re-use as structural backfill.

Measures should be taken to reduce the potential for build-up of hydrostatic pressure behind the retaining walls. Drainage design should include free-draining backfill materials and perforated drains as depicted on Figure 7. Solid outlet pipes should be connected to the perforated drains and then routed to a suitable area for discharge of accumulated water.

10.8. Preliminary Pavement Design Recommendations

During our field exploration, existing pavement sections and subgrade conditions were observed. In general, the pavement sections encountered consisted of asphalt concrete over subgrade soils. The thickness of the asphalt concrete throughout the site was observed to be approximately 4 to 5 inches thick at each of our boring locations.

The shallow subgrade soils encountered in our exploratory borings consisted of sandy clay. A representative, near-surface soil samples was tested for resistance value (R-value) in order to provide design pavement structural sections. Laboratory testing indicated the near surface sandy soils have an R-value of 69. However, due to the variability of the on-site soils, including the presence of clayey and silty soils, as well as our experience in the area, we used a design R-value of 40 for the purpose of developing asphalt pavement sections for the subgrade soils. No specific traffic loading information was available for our design at the time of the preparation of this report.

Based on our understanding of the project, we have anticipated that traffic will consist of mostly passenger-type, some heavy, and occasional emergency vehicles. Accordingly, we have assumed traffic indices (TI) of 5.0, 7.0, and 9.0 for the design of pavement sections. A TI of 5 has been assumed for general parking areas for passenger vehicles. A TI of 7 can be assumed for occasional emergency vehicles, and a TI of 9 can be assumed for areas with regular heavy truck traffic. Based on our design R-value and assumed traffic indices, we have developed the structural sections presented in Table 8 below.

Table 8 – Pavement Structural Sections

Traffic Index	Recommended Pavement Section	
	Flexible Pavement	
	AC/CAB (inches)	Full Depth AC (inches)
5	3.0/4.0	5.0
7	3.5/8.0	8.0
9	5.5/10.0	10.0
Notes: AC – Asphalt Concrete CAB – Crushed Aggregate Base		

In order to provide suitable support for the proposed pavement areas, we recommend that the subgrade soils be scarified approximately 12 inches, moisture conditioned to slightly over optimum moisture content, and compacted to 90 percent of the laboratory maximum dry density as evaluated by ASTM 1557. Crushed aggregate base should conform to the Standard Specifications for Public Works Construction “Greenbook,” Section 200. The crushed aggregate base material should be placed at a relative compaction of 95 percent. Asphalt concrete should conform to “Greenbook,” Section 203. We recommend that the paving operations be observed and tested by Ninyo & Moore.

10.9. Pre-Construction Survey and Monitoring

We recommend that a pre-construction survey of existing conditions be performed for adjacent buildings and properties prior to the start of construction. We also recommend that a monitoring program be implemented in order to monitor potential movement of existing buildings and properties and to help evaluate the vibration levels caused during construction related activities.

10.9.1. Documentation of Existing Conditions

The pre-construction survey should consist of photographic documentation of the exterior portions of adjacent buildings and the overall condition of adjacent properties,

including distress features such as cracks and/or separations that may be present. The purpose of the pre-construction survey is to develop documentation of existing conditions prior to construction that may serve as a basis for evaluating potential damage claims.

10.9.2. Construction Vibrations

People can perceive vibrations at low levels and tend to judge the vibrations as being much higher than they actually are. Hendron and Oriard (1972) stated that transient vibrations from construction activities, such as pile driving, are noticeable at peak particle velocities as low as 0.02 to 0.06 inch per second. At peak particle velocities as low as 0.2 to 0.4 inch per second, the vibrations are disturbing and may result in complaints and damage claims. However, these vibration levels are below the peak particle velocity threshold considered to cause cosmetic damage to commercial/residential construction.

We recommend that vibration caused by construction activities be monitored in terms of peak particle velocity during construction. To monitor the peak particle velocity, seismographs could be positioned at the construction site and monitored at selected intervals during construction. Consideration should be given to locating one seismograph close to the construction activity, another at the nearest building, and a third at a building farther away.

10.10. Corrosion

Laboratory testing was performed to evaluate soil pH, electrical resistivity, water-soluble chloride content, and water-soluble sulfate content of near-surface soil samples. The soil pH and electrical resistivity tests were performed in general accordance with California Test (CT) Method 643. Chloride content tests were performed in general accordance with CT Method 422. Sulfate content tests were performed in general accordance with CT Method 417. The laboratory test results are presented in Appendix C.

The results of the corrosivity testing indicated electrical resistivity ranging from approximately 114 to 154 ohm-centimeters, soil pH of 7.2 to 8.3, chloride contents varying between 3,270 and 6,240 parts per million (ppm), and sulfate content ranging from approximately 0.150 to 0.420 percent (i.e., 1,500 to 4,200 ppm). Based on the Caltrans (2003) criteria, the project site would be classified as corrosive, which is defined as a site having soils with more than 500 ppm of chlorides, more than 0.2 percent sulfates, or a pH less than 5.5.

10.11. Concrete Placement

Concrete in contact with soil or water that contains high concentrations of water-soluble sulfates can be subject to premature chemical and/or physical deterioration. The soil samples tested in this evaluation indicated water-soluble sulfate contents ranging from about 0.150 to 0.420 percent by weight (i.e., about 1,500 to 4,200 ppm). Based on the UBC criteria (ICBO, 1997), the potential for sulfate attack is negligible for water-soluble sulfate contents in soils ranging from about 0.0 to 0.10 percent by weight (0 to 1,000 ppm), moderate for water-soluble sulfate contents ranging from about 0.10 to 0.20 percent by weight (1,000 to 2,000 ppm), and severe for water-soluble sulfate contents ranging from about 0.20 to 2.00 percent by weight (2,000 to 20,000 ppm). Therefore, the site soils may be considered to have a moderate to severe potential for sulfate attack. Based on UBC criteria (ICBO, 1997), Type V cement should be used for concrete construction. The concrete should have a water-cement ratio no higher than 0.45 by weight for normal weight aggregate concrete and a 28-day compressive strength of 4,500 pounds per square inch (psi) or more.

In order to reduce the potential for shrinkage cracks in the concrete during curing, we recommend that the concrete for the proposed structures be placed with a slump of 4 inches based on ASTM C 143. The slump should be checked periodically at the site prior to concrete placement. We also recommend that crack control joints be provided in slabs in accordance with the recommendations of the structural engineer to reduce the potential for distress due to minor soil movement and concrete shrinkage. We further recommend that concrete cover over reinforcing steel for slabs-on-grade and foundations be provided in ac-

cordance with UBC 1907.7.1 (ICBO, 1997). The structural engineer should be consulted for additional concrete specifications.

10.12. Drainage

Proper surface drainage is imperative for satisfactory site performance. Positive drainage should be provided and maintained to direct surface water away from foundations and off site. Positive drainage is defined as a slope of 2 percent or more over a distance of 5 feet or greater away from structure foundations and top of slopes. Runoff should then be directed by the use of swales or pipes into a collective drainage system. Surface waters should not be allowed to flow over slope faces or pond adjacent to footings. Area drains for landscaped and paved areas are recommended. Nearby landscaping should consist of drought-tolerant plants, and landscape irrigation should be kept to a level just sufficient to maintain plant vigor. Overwatering should not be permitted.

11. CONSTRUCTION OBSERVATION

The recommendations provided in this report are based on our understanding of the proposed project and on our evaluation of the data collected based on subsurface conditions observed in one exploratory boring. It is imperative that the geotechnical consultant checks the subsurface conditions during construction. We recommend that Ninyo & Moore review the project plans and specifications prior to construction. It should be noted that, upon review of these documents, some recommendations presented in this report may be revised or modified.

During construction, we recommend that the duties of the geotechnical consultant include, but not be limited to:

- Observing clearing, grubbing, and removals.
- Observing excavation, placement, and compaction of fill.
- Evaluating imported materials prior to their use as fill (if used).
- Performing field tests to evaluate fill compaction.

- Observing foundation excavations for cleaning prior to placement of reinforcing steel.
- Field pile driving inspection to monitor and document the tip elevations and blow counts if driven piles are constructed.
- Field instrumentation during and after deep foundation construction to confirm the design assumptions.
- Performing material testing services including concrete compressive strength and steel tensile strength tests and inspections.

The recommendations provided in this report are based on the assumption that Ninyo & Moore will provide geotechnical observation and testing services during construction. In the event that the services of Ninyo & Moore are not utilized during construction, we request that the selected consultant provide the City of Long Beach with a letter (with a copy to Ninyo & Moore) indicating that they fully understand Ninyo & Moore's recommendations and that they are in full agreement with the design parameters and recommendations contained in this report.

12. LIMITATIONS

The field evaluation, laboratory testing, and geotechnical analyses presented in this geotechnical report have been conducted in general accordance with current practice and the standard of care exercised by geotechnical consultants performing similar tasks in the project area. No warranty, expressed or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be encountered during construction. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation will be performed upon request. Please also note that our evaluation was limited to assessment of the geotechnical aspects of the project, and did not include evaluation of structural issues, environmental concerns, or the presence of hazardous materials.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Ninyo & Moore

should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

This report is intended for design purposes only. It does not provide sufficient data to prepare an accurate bid by contractors. It is suggested that the bidders and their geotechnical consultant perform an independent evaluation of the subsurface conditions in the project areas. The independent evaluations may include, but not be limited to, review of other geotechnical reports prepared for the adjacent areas, site reconnaissance, and additional exploration and laboratory testing.

Our conclusions, recommendations, and opinions are based on an analysis of the observed site conditions. If geotechnical conditions different from those described in this report are encountered, our office should be notified, and additional recommendations, if warranted, will be provided upon request. It should be understood that the conditions of a site could change with time as a result of natural processes or the activities of man at the subject site or nearby sites. In addition, changes to the applicable laws, regulations, codes, and standards of practice may occur due to government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Ninyo & Moore has no control.

This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

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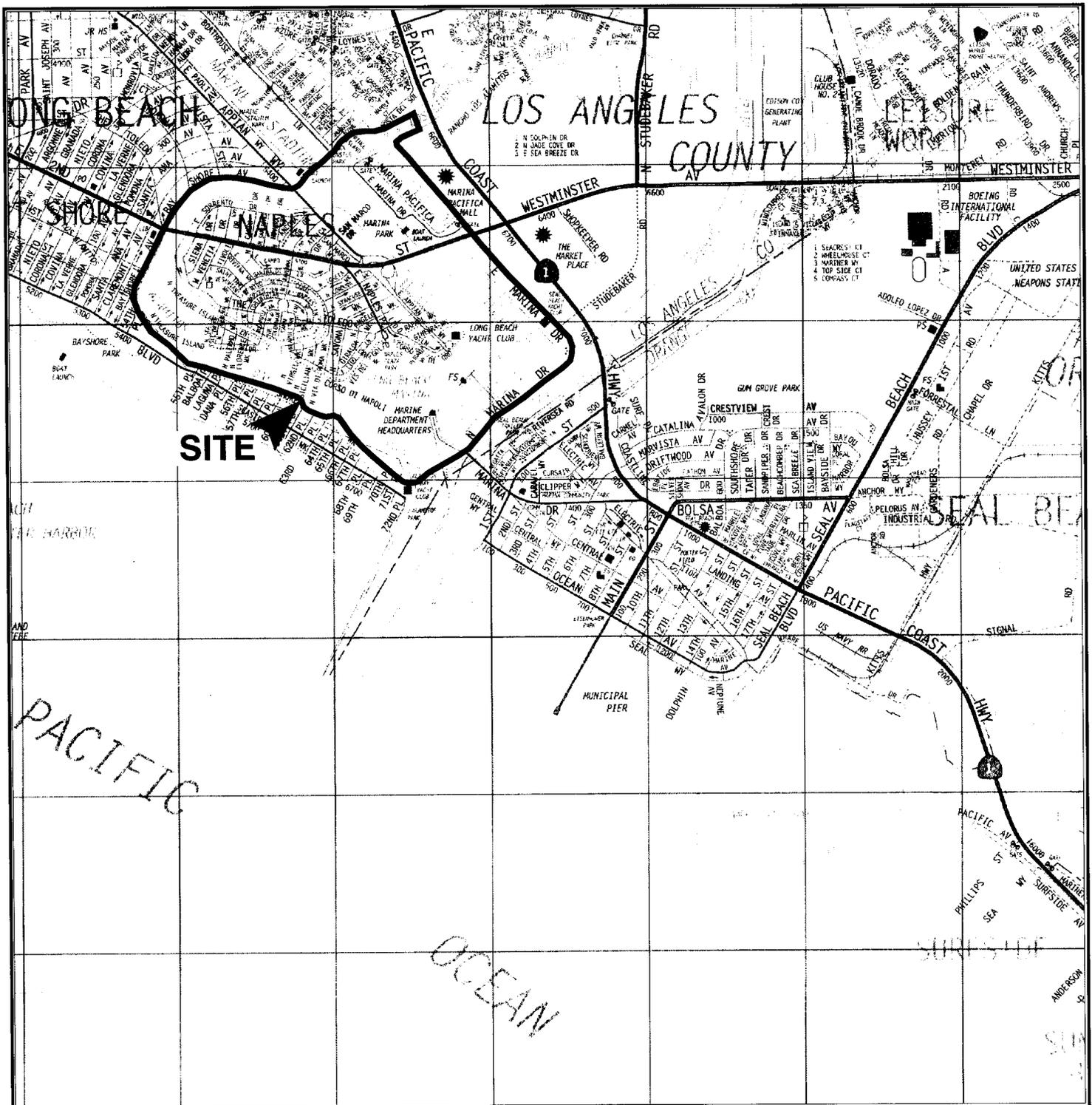
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Youd, T.L., Idriss, I.M., Andrus, R.D., Arango, I., Castro, G., Christian, J.T., Dobry, R., Finn, W.D., Harder, L.F., Hynes, M.E., Ishihara, K., Koester, J.P., Liao, S.S.C., Marcuson, W.F., Martin, G.R., Mitchell, J.K., Moriwaki, Y., Power, M.S., Robertson, P.K., Seed, R.B., and Stokoe, K.H., II., 2001, Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 124(10), 817-833.

Youd, T.L., Hanse, C.M., and Bartlett, S.F., 2002, Revised MLR Equations for Predicting Lateral Spread Displacement, Journal of Geotechnical and Geoenvironmental Engineering, Volume 128, Number 12, pp. 1007-1017, dated December.

AERIAL PHOTOGRAPHS				
Source	Scale	Date	Flight	Numbers
USDA	1:20,000	11-18-52	AXK-1K	3 and 4



REFERENCE: 2005 THOMAS GUIDE FOR LOS ANGELES/ORANGE COUNTIES, STREET GUIDE AND DIRECTORY

APPROXIMATE SCALE IN FEET



NOTE: ALL DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.



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SITE LOCATION MAP

FIGURE

PROJECT NO.

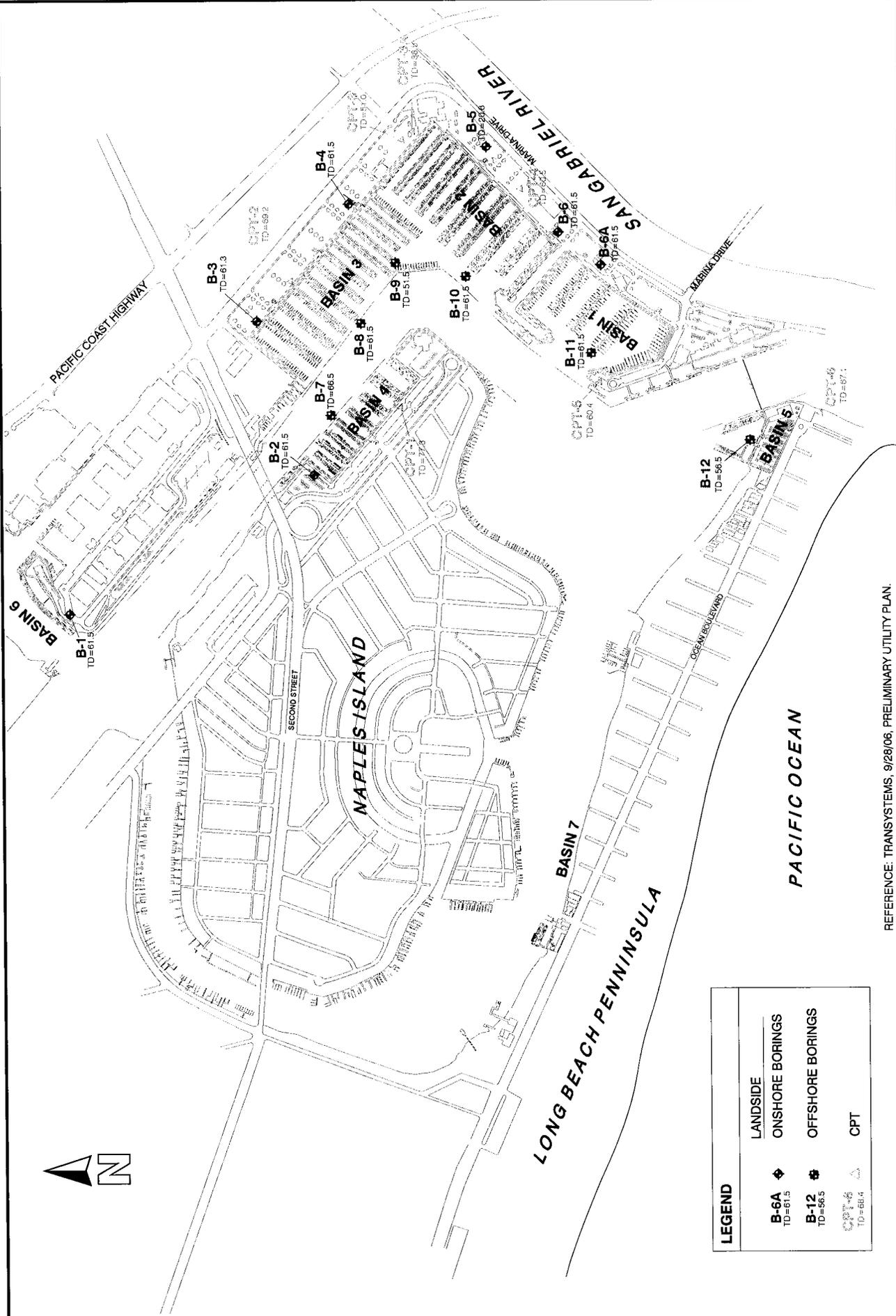
DATE

ALAMITOS BAY MARINA IMPROVEMENT PROJECT
LONG BEACH, CALIFORNIA

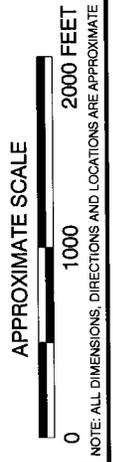
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1

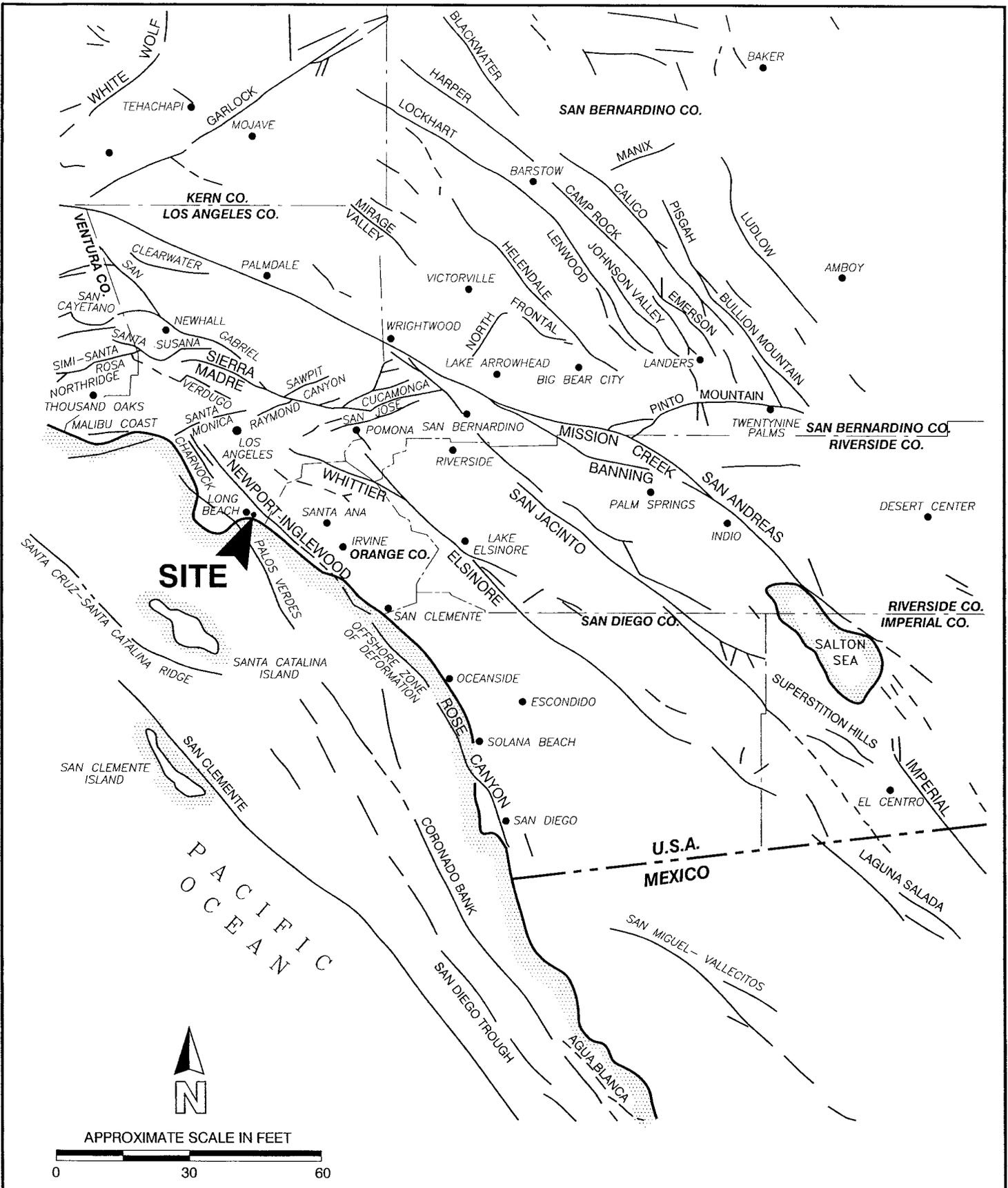


LEGEND	
	LANDSIDE
	ONSHORE BORINGS
	OFFSHORE BORINGS
	CPT



REFERENCE: TRANSYSTEMS, 9/28/06, PRELIMINARY UTILITY PLAN.

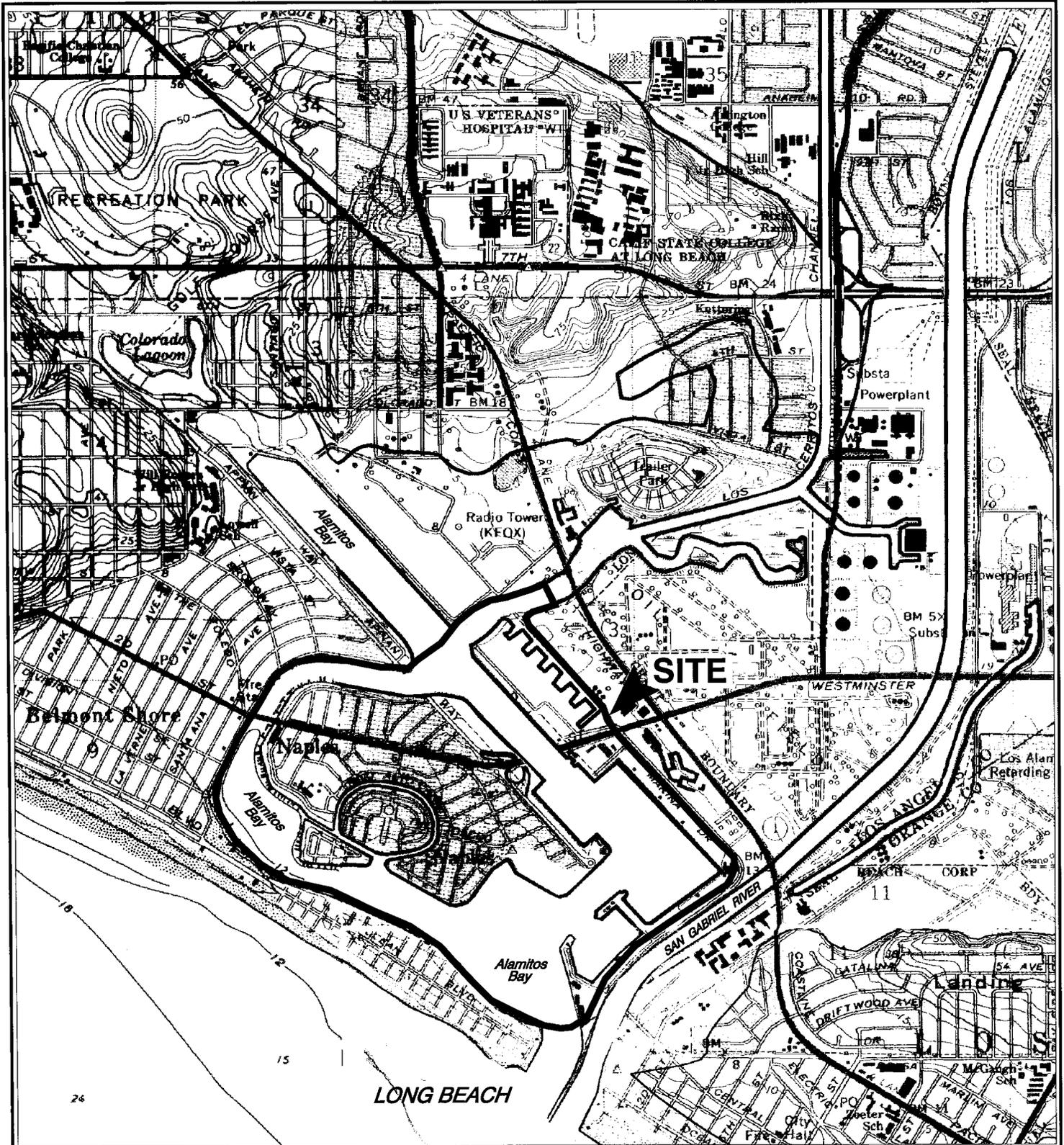
		BORING AND CPT LOCATION MAP		FIGURE 2
		ALAMITOS BAY MARINA IMPROVEMENT PROJECT LONG BEACH, CALIFORNIA		
PROJECT NO.	DATE			
206918001	2/07			



NOTE: ALL DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

		FAULT LOCATION MAP		FIGURE 3

E00003-B-16 (Rev. 11/00)



REFERENCE: STATE OF CALIFORNIA SEISMIC HAZARD ZONES LONG BEACH (1999), SEAL BEACH (1998), LOS ALAMITOS (1998) QUADRANGLE MAPS.

MAP EXPLANATION

LIQUEFACTION:
 Areas where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

APPROXIMATE SCALE IN FEET



NOTE: ALL DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.



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SEISMIC HAZARD ZONES MAP

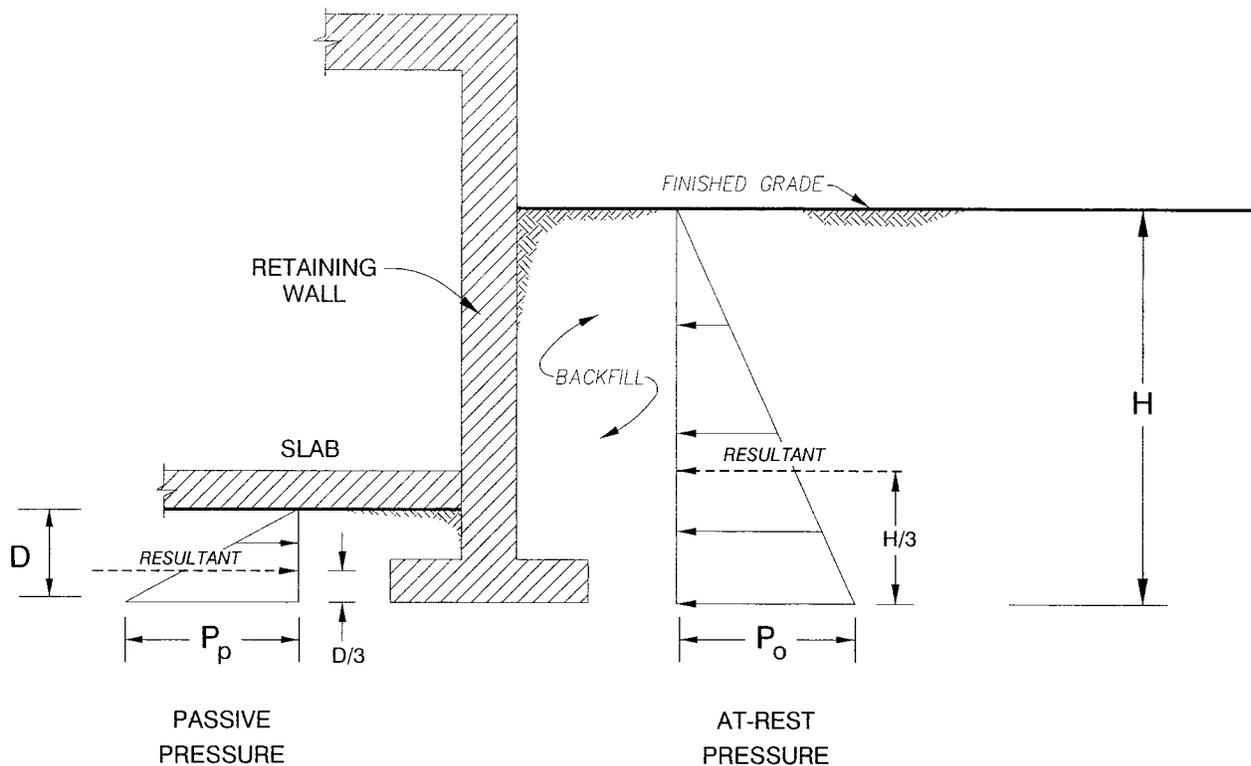
FIGURE

PROJECT NO. 206918001
 DATE 2/07

ALAMITOS BAY MARINA IMPROVEMENT PROJECT
 LONG BEACH, CALIFORNIA

4

206918-AZ1.DWG



NOTES:

1. ASSUMES NO HYDROSTATIC PRESSURE BUILD-UP BEHIND THE RETAINING WALL
2. STRUCTURAL, GRANULAR BACKFILL MATERIALS AS SPECIFIED IN SECTION 10.1 SHOULD BE USED FOR RETAINING WALL BACKFILL
3. DRAINS AS RECOMMENDED IN THE RETAINING WALL DRAINAGE DETAIL SHOULD BE INSTALLED BEHIND THE RETAINING WALL
4. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED
5. H AND D ARE IN FEET (H IS LESS THAN 12 FEET)

RECOMMENDED GEOTECHNICAL DESIGN PARAMETERS

Lateral Earth Pressure	Equivalent Fluid Pressure (lb/ft ² /ft) ⁽¹⁾	
	P _o	Level Backfill with Granular Soils ⁽²⁾
53H		82H
P _p	Level Ground	2H:1V Descending Ground
	300D	140D

NOT TO SCALE

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LATERAL EARTH PRESSURES FOR RESTRAINED RETAINING WALLS

FIGURE

PROJECT NO.

DATE

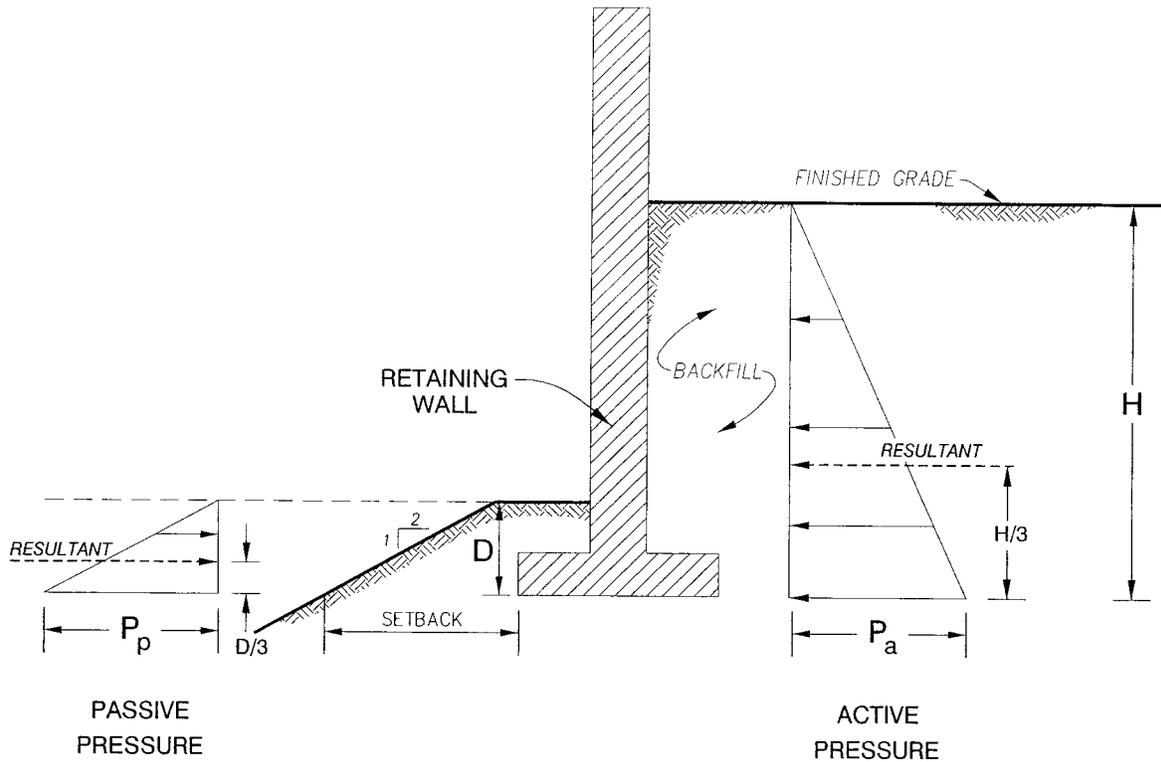
ALAMITOS BAY MARINA IMPROVEMENT PROJECT
LONG BEACH, CALIFORNIA

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206918001

2/07

206918_A18.DWG



NOTES:

1. ASSUMES NO HYDROSTATIC PRESSURE BUILD-UP BEHIND THE RETAINING WALL
2. STRUCTURAL, GRANULAR BACKFILL MATERIALS AS SPECIFIED IN SECTION 10.1 SHOULD BE USED FOR RETAINING WALL BACKFILL
3. DRAINS AS RECOMMENDED IN THE RETAINING WALL DRAINAGE DETAIL SHOULD BE INSTALLED BEHIND THE RETAINING WALL
4. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED
5. H AND D ARE IN FEET (H IS LESS THAN 12 FEET)
6. SETBACK SHOULD BE IN ACCORDANCE WITH FIGURE 18-I-1 OF THE CBC (2001)

RECOMMENDED GEOTECHNICAL DESIGN PARAMETERS

Lateral Earth Pressure	Equivalent Fluid Pressure (lb/ft ² /ft) ⁽¹⁾	
	Level Backfill with Granular Soils ⁽²⁾	2H:1V Sloping Backfill with Granular Soils ⁽²⁾
P_a	34H	53H
P_p	Level Ground	2H:1V Descending Ground
	300D	140D

NOT TO SCALE

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LATERAL EARTH PRESSURES FOR YIELDING RETAINING WALLS

FIGURE

PROJECT NO.

DATE

ALAMITOS BAY MARINA IMPROVEMENT PROJECT
LONG BEACH, CALIFORNIA

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206918001

2/07

**APPENDICES FOR THE GEOTECHNICAL EVALUATION FOR THE
ALAMITOS BAY MARINA REHABILITATION PROJECT
ARE ON FILE WITH THE CITY OF LONG BEACH MARINE BUREAU**