APPENDIX B

GEOLOGICAL ENGINEERING REPORT, ADDENDUM TO GEOLOGICAL ENGINEERING REPORT, AND GUIDE PILE MEMORANDUM

January 15, 2008

Project No. 600024-004

To: URS/Cash and Associates
5772 Bolsa Avenue, Suite 100
Huntington Beach, California 92649

Attention: Mr. Randy Mason, P.E.

Subject: Addendum to Geotechnical Engineering Report for the Proposed Renovations to Dana Point Harbor, City of Dana Point, California

The following letter is submitted in response to the conference call conducted on January 7, 2008 with the project team regarding the issue of liquefaction and the intent of the recommendations presented in the geotechnical report prepared by our firm (PN 600024-004, dated January 7, 2008) for the proposed renovation to Dana Point Harbor. This letter is considered to be an addendum to the geotechnical report and is intended to supplement the recommendations presented in the report.

Liquefaction Mitigation

As discussed in the geotechnical report, the soils that underlie the harbor and, in particular, the soils encountered below the Island and the peninsulas at the east and west ends of the marina, are susceptible to liquefaction in the event of strong ground shaking associated with an earthquake that has a 10 percent probability of occurrence in a 50-year exposure period. The results of liquefaction were estimated to include instability of the slopes that support the current seawall system, a short cantilever retaining wall that borders a slope that descends into the harbor in which the slope face is protected by concrete panels. The slope instability is expected to be manifested by lateral translation and associated distortion to the seawall system. Due to the fact that the soils that underlie the seawall system (retaining wall and slope panels) are fill materials, significant variation in the quality and compaction of the material should be expected to exit. The variability in the soil conditions may result in a corespondng variance in the effect of liquefaction on the seawall system.

Although the potential for liquefaction exists with potentially unfavorable effects, the necessity to implement remedial measures is left to the discretion of the client. The proposed renovations do not include the construction of any permanent structures that could be inhabited which would, therefore, require some form of remedial action though either the improvement of the subsoils to reduce liquefaction potential or design of foundation system that could resist the effects of liquefaction. If such types of structures are contemplated in the future or as part of the currently proposed renovation, the appropriate recommendations can be provided by our firm.

ADA Platform Foundations

The proposed renovation concept as currently proposed is not considered to have any effect on the liquefaction potential of the soils, i.e., the construction associated with renovation will not worsen or increase the potential for liquefaction. The installation of guide piles by pile driving, and the associated vibrations may initiate some consolidation of loose granular soils which could result on some settlement of improvements in close proximity to pile driving activities. Careful monitoring of the seawall or other improvements is recommended to be conducted when driving piles in close proximity. If distortions do occur, pile installation may need to be altered to consist of pre-dilling and setting the piles within pre-drilled boreholes.

The recommendations for the foundations of the platforms that are required for the ADA gangways include both shallow and deep alternatives. The selection of the appropriate foundation was considered to be dependent upon the desired level of serviceability should liquefaction occur. The shallow foundation alternate consisted of a mass-concrete pour in which the concrete mass would be of the same plan dimension as the proposed platform with the bottom supported a minimum of 7 feet below grade. Based upon out understanding of the size of the platform (approximately 8 x 10 feet), the volume of the excavation (and concrete mass) would be approximately 21 yd^3. The deep foundation was recommended to consist of a drilled pier foundation extended an adequate depth below grade to resist the lateral forces associated with slope instability and maintain serviceability of the platform. The actual depth of embedment is to be determined on the basis of structural analysis considering the magnitude of the lateral soil surcharge force and the available resistance of the soils and bedrock below the plane of potential slope instability.

The drilled pier foundation may be substituted by the pile foundations where installed in a pre-drilled borehole to the required depth. Installation of the piles by driving should be use with caution as previously discussed relative to the potential for initiating some consolidation of the loose granular soils and settlement to the wall system.

Wall Surcharges

The construction of the ADA platforms will require the use of potentially heavy equipment to perform the construction activities. The weight of vehicles in close proximity to the wall could result in load surcharges that may result on distortion to the wall. Geotechnical design parameters and recommendations were presented in the report that allow the static stability of the wall and the effect of such surcharges to be evaluated.

Closing

We sincerely appreciate this opportunity to be of continued service. We hope that the discussion presented herein clarified the recommendations presented n the report with respect to the liquefaction potential of the soils and the considerations for future development. If you have any questions regarding this letter, please do not hesitate to contact this office.

Respectfully submitted,

LEIGHTON CONSULTING, INC.

John E. Haertle, PE, GE 2352
Senior Project Engineer

JEH/lr

Distribution: (2) Addressee
(2) LSA Associates, Inc.
Ms. Ashley Davis

GEOTECHNICAL ENGINEERING EXPLORATION AND ANALYSIS FOR THE PROPOSED RENOVATIONS TO DANA POINT HARBOR, CITY OF DANA POINT, CALIFORNIA

Prepared for:

URS/Cash and Associates
5772 Bolsa Avenue, Suite 100
Huntington Beach, California 92649

Project No. 600024-004

January 7, 2008

Leighton Consulting, Inc.

A LEIGHTON GROUP COMPANY

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January 7, 2008

Project No. 600024-004

To: URS/Cash and Associates
5772 Bolsa Avenue, Suite 100
Huntington Beach, California 92649

Attention: Mr. Randy Mason, P.E.

Subject: Geotechnical Engineering Exploration and Analysis for the Proposed Renovations to Dana Point Harbor, City of Dana Point, California

In accordance with your request and authorization, Leighton Consulting, Inc. has conducted a geotechnical engineering exploration and analysis for the renovations proposed for the Dana Point Harbor in the city of Dana Point, California. The exploration and analysis was conducted to allow evaluation of the existing seawall under static and seismic conditions, and provide recommendations for the design and construction of pedestrian platform structures and guide piles within the marina for new permanent and temporary boat docks.

The results of the current study indicate the seawall along the northern boundary (Cove region) of the harbor may be considered to be generally stable with respect to the overall rotational or gross stability in spite of the presence of layers of soils that exhibited the susceptibility to liquefaction. The potential for liquefaction was considered to be of greater significance along the southern seawall (Island region) and the small peninsular areas along the Sport Fishing Docks in the eastern region of the harbor and the Youth & Group facility in the western region of the harbor. The potential for liquefaction to occur may result in slope instability and associated lateral displacements.

The performance requirements of the new guide piles are expected to be primarily influenced by the lateral load capacity of the piles. The piles are recommended to be installed to a depth below the submarine bedrock surface a distance of 10 to 15 feet. Pile installation by in-situ construction techniques is understood to be the preferred manner of construction to minimize noise and vibration disturbances to the occupants and residents in the area as compared to pile driving. The installation of the piles is, however, expected to encounter some difficulties due to the presence of oversize material such as boulders or rock slabs that posed difficulty in advancing some of the test borings and CPT soundings to planned depth.

The foundations for the pedestrian platforms at the ADA Gangways are recommended to consist of drilled piers in areas where the liquefaction potential of the subsoils could result lateral translation if the serviceability of the platforms after liquefaction is desired. If the risk of damage is acceptable, the platforms may be supported by a deepened, mass-concrete spread footing foundation or shallower drilled piers to transfer lad below the depth at which the adjacent seawall bears to avoiding lateral load surcharges on the wall.

The report provides additional details regarding the subsurface conditions and recommendations for design and construction.

If you have any questions regarding this report, please do not hesitate to contact this office. We appreciate this opportunity to be of service.

REGISTERED PROFESSIONAL ENGINEER
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No. 2352
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Respectfully submitted,

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1.0 INTRODUCTION

1.1 Site Location and Description

The proposed project consists of renovation of the Dana Point Harbor, the location of which is shown in Figure 1 included at the end of the text. The renovation will include reconstruction of the boat slips within the marina, improvements to the docks at the Embarcadero, Shipyard, Sport Fishing and the Harbor Patrol in the eastern region of the harbor; new docks at the Youth and Group facility in the western region; new ADA access ramps at six locations within the harbor; and new temporary docks along the south and west sides of the Island and along the eastern breakwater. The proposed improvements are shown in Figure 2.

The Dana Point Harbor was constructed in the late 1960s by the County of Orange and The United States Army Corps of Engineers. The current capacity of the harbor is approximately 2,400 boat slips. The northern half of the harbor is referred to as the Cove region while the southern half is referred to as the Island region. The northern and southern regions are divided into eastern and western basins by a bridge (Island Way) that provides vehicle access from the Cove side of the harbor to the Island side, thereby dividing the harbor into approximate quadrants. The harbor is protected from wave action of the Pacific Ocean by a breakwater south of the island.

Information provided by personnel familiar with the historical development of the harbor indicates the harbor was constructed by excavation of the basins after initially dewatering through the construction of a coffer dam. The construction of the coffer dam is understood to have included the installation of sheetpiling and initial filling with earth materials placed in-the-wet. The basins were originally planned to be excavated to an elevation (El.) of 10 feet below mean low level water (MLLW). However, due to the hardness of the bedrock material, the northwestern quadrant was only excavated to approximately El. -8 feet. Verbal accounts provided by individuals familiar with the initial construction of the harbor indicated significant pile driving difficulties were encountered. Pile driving ultimately required the use of a "stinger," a steel beam spliced to the end of a square concrete pile.

1.2 Existing Seawall System

A structural evaluation of the seawall system was previously conducted by Bluewater Design Group (Bluewater, 2003). Structural details of the existing wall system were obtained from review of the referenced report. Review of the report indicates the seawall system along the north and south sides of the basins of the harbor consists of a cantilever retaining wall that is located at the crest of a slope that descends at an inclination of 1.5H:1V into the adjacent basins. The face of this descending slope is covered by a revetment which consists of a series of cast-in-place concrete panels that are approximately 10 feet wide and 20 feet in length. The panels have been reported to be approximately 6 inches in thickness. The panels are typically situated such that a gap of approximately 1½ inches exists between successive panels.

The seawall system that exists in the boat launch area and along the eastern access channel of the marina generally consists of a similar retaining wall with rock rubble revetment covering the descending slope. The retaining structure along the south side of the boat launch ramp consists of a cantilever retaining wall of varying height.

The retaining walls along the northern and southern sides of the basins consist of a conventional cantilever wall that is approximately 5 feet in height. The wall foundation was reported to be constructed as cast-in-place concrete with the wall consisting of pre-cast panels. The typical wall section indicates the wall stem is slightly battered. Due to the lack of footing embedment and the proximity to a descending slope, the design of the wall relative to lateral sliding stability was based solely on sliding friction acting on the base of the footing. Load transfer to the revetment panels is precluded by the as-designed 1½-inch wide gap between the wall footing and the panels.

1.3 Purpose and Scope

This report presents the results of the geotechnical study conducted by our firm for the improvements planned for Dana Point Harbor. The primary focus of our evaluation was the stability of the seawall under seismic conditions; the construction of new guide piles that will be required for permanent and temporary docks; and the foundations to support the platforms for the ADA gangways. The scope of our work included the following tasks:

- Review of available geologic reports and maps.
- Notification of Underground Service Alert (USA) of marked boring locations prior to the commencement of our field exploration and coordination of a drilling contractor.
- Subsurface exploration consisting of excavation, logging, and sampling of nine (9) hollow-stem auger borings and ten (10) Cone Penetrometer Test (CPT) soundings. The initial phase of exploration was conducted at locations pre-selected by the client, while the subsequent phase was conducted at locations selected by the project engineer based upon the proposed construction.
- Collection of relatively undisturbed and bulk soil samples at selected depth intervals from the soil borings and transportation of the samples to our laboratory for testing.
- Laboratory testing of selected samples to evaluate engineering characteristics of the onsite earth materials within the exploration depths.
- Geotechnical evaluation of collected test boring and CPT data and relevant engineering analyses.
- Preparation of this report summarizing our findings, conclusions, and recommendations.

2.0 FIELD EXPLORATION AND LABORATORY TESTING

2.1 Field Exploration

Prior to the field explorations conducted in January and then later in November 2007, a cursory visual site reconnaissance was performed by a geotechnical engineer from our staff to review the locations of the proposed test borings and CPT soundings and to evaluate the marked locations with respect to access for heavy equipment. Underground Service Alert of Southern California (USA) was then notified of the marked locations so that known utilities could be indicated on the ground surface. In addition to utility location by USA, Geovision, Inc. was retained to provide geophysical testing to further identify underground utilities.

The locations of the soil borings and CPT soundings are shown in Plate 1, Boring and CPT Exploration Map, included (in pocket) at the end of the report and on Figure 3, which is reduced version of the oversize Plate.

January 2007: Four (4) soil borings (B-1 through B-4) were drilled at the pre-selected locations during the period of January 11 and 12, 2007. The boring locations were initially selected by the client for the primary purpose of evaluating the stability of the existing seawall. The borings were advanced to depths that ranged from 34 to 50 feet below the current grade by a truck-mounted drilling rig using rotary drilling techniques. All borings were extended into the formational bedrock that underlies the area.

In addition to the soil borings, five (5) CPT soundings were performed during the same time frame as the test borings. It should be noted that CPT-5 was located adjacent to CPT-2 in an effort to explore conditions to a greater depth than was achieved at CPT-2. For purposes of this report, the data from CPT-5 has been used in place of CPT-2; references to CPT-2 are actually the data collected from CPT-5. The CPT soundings were situated in close proximity to the boring locations so that samples collected from the test boring locations could be correlated to the CPT data and the profile penetrated by the CPT probe. The CPT soundings met with refusal at depths of 12½ to 33 feet below grade. Based upon the boring data, all CPT soundings except for CPT-4 were terminated in the formational bedrock while CPT-4 was presumed to have been terminated on cobble or boulder material.

November 2007: Five (5) soil borings (B-5 through B-9) were drilled during the period of November 5 and 6, 2007 at locations that were selected on the basis of the proposed improvements. The borings were advanced to depths that ranged from approximately 26½ to 55½ feet below current grade by a truck-mounted drilling rig using rotary drilling techniques. All borings with the exception of Boring B-6 were extended into the formational bedrock that underlies the area. Boring B-6 (south of the Yacht Club) was terminated at a depth of approximately 26½ feet due to drill rig auger refusal on possible cobble and boulder or rock slab material.

Exploration by soil borings was supplemented by advancing five (5) CPT soundings (CPT Nos. 6 through 10) during the same time period as the test boring exploration. The CPT soundings were situated in close proximity to the boring locations so that samples collected from the test boring locations could be correlated to the CPT data and the profile penetrated by the CPT probe. The CPT soundings met with refusal at depths of 16½ to 35½ feet below grade. Based upon the boring data, all CPT soundings except for CPT-6 and -7 were terminated in the formational bedrock where CPT-6 and -7 were presumed to have been terminated on cobble or boulder material.

Equipment and Procedures: The test borings were advanced by a conventional truck-mounted drill rig using hollow-stem flight augers. Drilling fluid was added to the drill stem to provide a counteractive effect to unbalanced hydrostatic pressures that could develop due to the depth of the borings below the water table. The rig was equipped with an automatic-trip hammer that ensures a consistent hammer drop during sampling and field testing. All borings were continuously logged during drilling by a member of our technical staff. Samples were collected at selected intervals using spilt-barrel sampling methods in accordance with ASTM D1586 and D3550 specifications. Each sample collected was classified in the field in general conformance with the Unified Soil Classification System (USCS) and subsequently reviewed in the laboratory. All samples were sealed and packaged for transportation to our laboratory for testing.

Field testing conducted during soil sampling consisted of driving the sampler below the bottom of the borehole with successive drops of a 140-pound weight falling 30 inches. The number of blows (blow counts) to drive the sampler 18 inches was recorded for each 6-inch increment of penetration. The blow counts were recorded on the logs of the test borings. In addition, bulk samples of the soils were collected from selected depths. Upon completion of the drilling, the boreholes were backfilled with soil cuttings and the boreholes patched.

The CPT soundings were performed in accordance with ASTM D5778 and D3441 specifications. The test boring logs and the logs of the CPT soundings are included in Appendix B.

2.2 Seafloor Profiling

Field exploration has also included the use of specialized geophysical testing to conduct a survey of the seafloor bottom throughout the harbor. The profiling provides a generalized description of the seafloor bottom as well the contact between seafloor sediments and the underlying bedrock or other materials that provide a contrast in density. The results of the survey were not yet available at the time of geotechnical analysis. An addendum to this report will be prepared upon receipt of the survey and review of the interpretation of the findings.

2.3 Laboratory Testing

Laboratory tests were performed on selected samples to verify the field classification of the recovered samples and to determine the geotechnical engineering properties of the subsurface materials. Laboratory testing was performed to evaluate the following engineering properties of the soils and bedrock:

- Determination of the in-situ moisture content and density;
- Particle size distribution by mechanical sieve analysis and hydrometer sedimentation;
- Determination of fines content by Percent Passing No. 200 Sieve;
- Shear strength by Direct Shear, Unconfined Compression and Triaxial Compression testing;
- Soil plasticity by determination of Atterberg Limits; and
- Soil corrosivity as indicated by the concentration of water soluble sulfate, minimum resistivity, chloride concentration and pH.

All laboratory tests were performed in general conformance with ASTM Standard Test Methods. The results of the in-situ moisture content and density are included on the logs of the test borings while the results other tests are presented in Appendix C of this report.

3.0 SUBSURFACE CONDITIONS AND SEISMICITY

3.1 Regional Geology

3.1.1 Geologic Setting

Dana Point Harbor is located within the northwest trending Peninsular Ranges geomorphic province in southern California. The Peninsular Ranges province is elongated area characterized by parallel fault-bounded mountain ranges and intervening valleys. The province extends southward from the Transverse Ranges at the northern side of the Los Angeles Basin southward into Mexico. The site lies at the southernmost end of The San Joaquin Hills, which are a northwest trending topographically elevated area that extend southward from Newport Beach to Dana Point.

The harbor is a coastal reentrant or cove protected by the headland at Dana Point. The protected cove owes its existence to the differing resistance to wave erosion of the two bedrock formations exposed along a fault in the steep coastal bluff. Bedrock units include the Capistrano Formation and the San Onofre Breccia. Both the San Onofre Breccia and the Capistrano Formation are exposed in the sea cliffs behind the harbor where they are separated by the Dana Cove Fault. However, we only encountered the Capistrano Formation during our subsurface investigation. The weaker Capistrano Formation has been preferentially eroded, creating Dana Cove. More youthful sediments have been deposited in the harbor including colluvium, alluvium, beach deposits, landslide debris, talus, and artificial fill placed during construction of the modern harbor in 1969 and 1970.

3.1.2 Bedrock Units

San Onofre Breccia: The San Onofre Breccia is a Middle Miocene-age (about 11 to 16 million years old) formation of marine origin. It consists of a very coarse, reddish-brown to blue-gray, massive to crudely bedded breccia with interbeds of coarse, pebbly sandstone and siltstone. The matrix is generally an earthy, poorly cemented silt, or a well-cemented angular sand. The San Onofre Breccia is exposed at the western end of Dana Point Harbor along the east-facing sea cliffs where it is in fault contact with the Capistrano Formation. The San Onofre Breccia is a bedrock unit that is resistant to erosion and forms the headland at Dana Point.

Capistrano Formation: The Capistrano Formation is a Late Miocene to Early Pliocene-age (about 3.6 to 11 million years old) formation of marine origin. In the Dana Point area, the Capistrano Formation is widespread with a total thickness of nearly 2,400 feet (Eddington, 1974). This marine (ocean deposited) bedrock formation is divided into a few recognizable subunits: a siltstone facies, a sandstone facies, and sandstone with conglomerate and sedimentary breccia. These three facies of the Capistrano Formation are all exposed in the sea cliffs surrounding the subject site generally dipping into slope (north). The siltstone facies is medium to dark gray and brownish gray to dark greenish gray, fine grained, poorly to moderately consolidated and massive to moderately fissile (Eddington, 1974). The sandstone facies is yellowish brown to pale yellowish brown and medium gray to light gray, fine to medium grained and weakly cemented, and massive to poorly bedded (Eddington, 1974). The sandstone and breccia facies is yellowish brown and coarse grained, weakly cemented to friable, with angular to rounded pebbles and cobbles of multiple origins, massive to poorly bedded, and with interbeds of well-graded sand and silt (Eddington, 1974). The bedrock encountered in the borings are from the siltstone facies of the Capistrano Formation. Capistrano Formation bedrock adjacent to the Dana Cove fault contact is sheared in a zone about 70 to 100 feet wide (Kerwin, 1992).

3.2. Subsurface Soil Conditions

The subsurface conditions that are described below have been summarized for clarity. Specific descriptions of the materials encountered at the boring and CPT sounding locations can be found on the logs presented in Appendix B. The boring logs provide the subsurface stratigraphy relative to the grade at the time of exploration and the corresponding elevations, which were referenced to the site grades shown on the base map provided by the client that was used as the basis for our exploration map (Plate 1, Figure 3). The elevations shown on the base map have been assumed to be relative to Mean Sea Level (MSL).

Existing Fill: The results of the field exploration indicate the presence of fill to depths that varied from approximately 10 to 20 feet on the Cove side of the harbor to depths of approximately 23 to 30 feet below the Island side of the harbor. The fill that underlies the Cove side of the harbor typically consisted of fine to medium grained sands with varying clay content that exhibited loose to medium dense relative density on the basis of field testing (N-values). The fill material encountered below the Island side of the harbor also consisted primarily of sand with greater silt and occasional clay content. The fill generally exhibited loose to medium dense relative density on the basis of field testing. Field tests that indicated dense relative density (N-values greater than 30) are not considered to be indicative of the actual relative density of the material but rather due to the influence of the presence of oversize (cobble and boulder) material within the fill.

Native Alluvial Soils: At several boring locations, the fill was underlain by native soils comprised of loose relative density sands with varying clay content to a depth of about 17 to 25 feet on the Cove side of the harbor. Native soils were generally not identified at the boring located on the Island side of the Harbor. Possible alluvial material was, however, indentified at Boring B-3 at a depth of about 30 feet below grade.

Bedrock: The bedrock was encountered at depths of 17 to 25 feet below grade in the Cove region of the harbor and at greater depths below the Island region. The bedrock contact appeared to be shallower along the north side of the Island where bedrock was encountered at depths of 23 to 28 feet as compared to the south side of the Island where bedrock was encountered and at a depth of 37 feet at Boring B-7.

Bedrock of the Capistrano formation was encountered below the fill and native soils at the depths described above. The bedrock typically consisted of interbedded layers of sandstone and siltstone.

3.3 Groundwater Conditions

Groundwater was typically encountered at depths of 9 to 16 feet below grade at the test boring locations during field exploration. The groundwater table was, however, estimated to exist at depths of 6 to 10 feet below grade on the basis of the relative moisture contents of the recovered soil samples. Groundwater in the areas of the seawalls is, however, expected to be subject to tidal fluctuation.

3.4 Faulting and Seismicity

3.4.1 Faulting

Two major faults are located in close proximity to the site. A description of these faults is presented below:

Newport-Inglewood Fault Zone: The Newport-Inglewood Fault Zone is a broad zone of left-stepping en echelon faults and folds striking southeastward from near Santa Monica across the Los Angeles basin to Newport Beach. Altogether these various faults constitute a system more than 150 miles long that extends into Baja California, Mexico. Faults having similar trends and projections occur offshore from San Clemente and San Diego (the Rose Canyon and La Nacion Faults). A near-shore portion of the Newport-Inglewood Fault Zone was the source of the destructive 1933 Long Beach earthquake (M_L 6.3) (Hauksson and Gross, 1991). This fault zone is considered a Type B fault (CDMG, Peterson et al., 1996), and the reported recurrence interval for a large event along this fault zone is 1,200 to 1,300 years with an expected slip of 1 meter (Forest et al., 1997).

San Joaquin Hills Blind Thrust Fault: The seismic hazards in Southern California have been further complicated with the recent realization that major earthquakes can occur on large thrust faults that are concealed at depths between 5 to 20 km, referred to as "blind thrusts." The uplift of the San Joaquin Hills is produced by a southwest dipping blind thrust fault that extends at least 14 km from northwestern Huntington Mesa to Dana Point and comes to within 2 km of the ground surface (Grant et al., 1997; Mueller et al., 1998). Work by Grant et al. (1997 and 1999) suggest that uplift of the hills began in the Late Quaternary and continues during the Holocene. Uplift rates have been estimated between 0.25 and 0.5 mm/yr. If the entire length of the fault ruptured, the earthquake generated has been estimated to be M_w 6.8 (Grant et al., 1999).

3.4.2 Ground Motion

The site is likely to experience strong ground shaking during the life of the development. Peak horizontal ground acceleration (PHGA) is generally used to characterize the amplitude of ground motion. A probabilistic seismic hazard analysis (PSHA) was performed using FRISKSP (Blake, 2000) and the recently published fault data (Cao, et al., 2003 and Peterson, et al., 1996) to estimate the PHGA value at the site for all active or potentially active faults from results of our search within a 100-kilometer radius of the site. This approach takes into account site-specific response characteristics, historical seismicity, and the geological characteristics of all faults under consideration. Three attenuation relationships were used in this analysis: Bozorgnia et al, 1999, Campbell et al., 1997 rev., and Sadigh et al., 1997. The results are presented in Appendix D.

Based on our probabilistic seismic hazard analysis, the results suggest that the estimated PHGA with a 10 percent probability of exceedance in 50 years is approximately 0.38 (recurrence interval of 475 years) for the site.

3.5 Seismic Hazards

The potential hazards to be evaluated with regard to seismic conditions include fault rupture, soil liquefaction, earthquake-induced vertical and lateral displacements, landslides triggered by groundshaking, earthquake-induced flooding due to the failure water containment structures, seiches, and tsunamis. An evaluation of these effects on the marina was previously discussed in our geotechnical report (Leighton, 2002) for preliminary planning purposes associated with Dana Point Master Plan. The following discussion is limited to the seismic factors associated with seawall system. The primary seismic hazard associated with the seawall is the potential for liquefaction and the potential for slope instability.

3.5.1 Fault Rupture

The harbor is not located within a currently designated Alquist-Priolo Earthquake Zone (Hart and Bryant, 1999). No known active faults are mapped on the site. Based on this consideration, the potential for surface fault rupture at the site is considered to be low.

3.5.2 Liquefaction

Liquefaction is a seismic phenomenon in which loose, saturated, non-cohesive granular soils exhibit severe reduction in strength and stability when subjected to high-intensity ground shaking. The mechanism by which liquefaction occurs is the progressive increase in excess pore pressure generated by the shaking associated with seismic event and the tendency for loose non-cohesive soils to consolidate. As the excess pore fluid pressure approaches the in-situ overburden pressure, the soils exhibit behavior similar to a dense fluid with a corresponding significant decrease in shear strength and increase in compressibility. Liquefaction occurs when three general conditions exist: 1) shallow groundwater; 2) low density, non-cohesive sandy soils; and 3) high-intensity ground motion. Studies indicate that saturated, loose and medium dense, near-surface cohesionless soils exhibit the highest liquefaction potential, while dry, dense, cohesionless soils exhibit low to negligible liquefaction potential. Bedrock and cohesive fine-grained soils are not considered susceptible to liquefaction.

The proposed project site is located in an area that has been identified by the State of California as being potentially susceptible to liquefaction, thereby requiring a site-specific evaluation of the potential for liquefaction to occur and appropriate remedial measures commensurate with the proposed structure.

The potential for liquefaction to occur has been evaluated based primarily upon the subsurface data collected by the CPT sounding exploration and the software package *LiquefyPro* (CivilTech, 2003). The procedure used in the software to determine the potential for triggering of liquefaction was the empirical procedure described by Robertson and Wride as adapted by the NCEER (1998; Youd 2001). The liquefaction analysis was conducted on the basis of a seismic event (M_w) of 6.8 and peak horizontal ground acceleration (PHGA) of 0.38g. The analysis was based upon a minimum Factor of Safety of 1.1 for liquefaction triggering.

The results of the liquefaction analysis are included in Appendix D which consists of the graphical output of the computer program. The analysis was focused on the CPT soundings rather than the test borings due to the refinement offered by the continuous record of the subsurface profile provided by the data collected during the penetration of the CPT probe. Analysis was not conducted for CPT-6 and CPT-4 since the conditions encountered at these CPT locations were not considered to be representative of the actual subsurface profile due to the apparent premature termination of the CPT soundings on oversize material as evidenced by comparison the of the interpreted CPT profiles with the adjacent borings. Liquefaction analysis at CPT-4 was substituted by the recently conducted CPT-9 while CPT-6 was substituted by Boring B-5.

The liquefaction analysis indicated the potential for liquefaction to occur within the fill and alluvial soils that comprise the Island as well the small peninsula adjacent to the Sport Fishing Docks in the eastern region of the harbor and in the peninsula area of the Youth & Group facility in the western region of the harbor. Liquefaction potential was determined to exist in either relatively thin layers or significantly thicker zones, typically on the order of 10 to 15 feet in thickness. The liquefaction potential was found to significantly less extensive in the Cove side of the harbor.

3.5.3 Seawall Global Stability

The occurrence of liquefaction is expected to primarily result in the reduction in slope stability due to the presumption that the subsurface profile consists of generally continuous horizontal layers of alluvial/sea floor deposits and fill materials. Liquefaction within layers that are assumed to be continuous below the seawall and extend into the adjacent basin results in predetermined planes or zones of weakness along which instability may occur.

The stability of the slopes that support the retaining wall of the seawall system was analyzed using the software package *GSTABL7 with STED* (Gregory, 2003). The program is capable of performing a search routine to determine the potential slip surface with the lowest Factor of Safety. The subject slopes were analyzed using Block Glide and Circular slip surface models. The Block Glide model allows the analysis to consider the shear strength of specific horizontal planes or other planes of geologic discontinuity. In the case of the seawall stability, the plane of weakness would be a continuous horizontal layer of liquefied soils. The shear strength characteristics used in the analysis were determined by laboratory testing of representative samples of the fill and bedrock material for the layers that were determined to not be susceptible to liquefaction.

The strength parameters used in the analysis for soil layers that were determined to be susceptible to liquefaction were based upon the recommendations presented by Seed and Harder (1990; SCEC, 1999) and Stark and Mesri (1992). The methodology to estimate the post-liquefaction residual undrained shear strength for slope stability analysis requires the correlation of the field N-value, corrected for overburden pressure, hammer efficiency, fines content, etc. [$(N1_{60})_{cs}$], to the residual undrained shear strength. The correlation was performed using the field N-values obtained by field sampling of the test borings and the equivalent N-values derived from CPT data based upon published correlations. The slope stability analysis is presented in Appendix E for the subsurface profile determined at the locations of the CPT-1/Boring B-1, CPT-2/Boring B-2, and CPT-3/Boring B-3 and CPT-10/Boring B-8 based upon the typical seawall system described in the preliminary structural evaluation (Bluewater 2003).

A minimum Factor of Safety of 1.25 was considered to be appropriate for the analyzed conditions, i.e., the relatively short time frame in which the residual undrained shear strength is representative of the potentially liquefiable strata.

Upon completion of excess pore pressure dissipation, the static, non-liquefiable strengths are considered to be re-established and the stability of the seawall system is increased. Based upon the results of the analysis, slope instability appears to be of significance for the profiles analyzed in the western region of the Island (CPT-3/Boring-3) and the peninsula area adjacent to the Sport Fishing Docks (CPT-10/Boring B-8). The similarity in profiles and results of the liquefaction triggering analysis also indicates the potential for slope instability in the peninsula at the Youth & Group facility. Due to the fact that the Island consists of reclaimed land, significant variance in the subsurface conditions should be expected to exist. The conditions encountered at CPT-3/Boring B-3 and farther south at CPT-8/Boring B-7 are anticipated to be representative of the majority of the Island unless further field exploration is performed to better define the subsurface profile. The results of the analysis are presented in Appendix E.

3.5.4 Seismically-Induced Slope Displacements

The occurrence of liquefaction and the potential for slope instability at the referenced locations indicates lateral displacement of the seawall system is likely through the phenomenon of Lateral Spreading. Evaluation of the potential displacements that may occur due to lateral spreading was performed using the empirical procedure developed by Youd, Hansen and Bartlett (1999) for the Free Face condition. Estimation of the potential lateral displacement by the empirical method suggests displacements on the order of several feet for the referenced profiles. In consideration of the adequate factor of safety for the remaining analyses, the potential for lateral spreading is considered to be low along the Cove side of the harbor, although variance in soil conditions in areas not previously explored as part of this study may result in different effects on wall stability and distortions.

3.5.5 Earthquake-Induced Settlements

Earthquake-induced settlements will consist of dynamic settlements (above groundwater) and liquefaction settlements (below groundwater). These settlements occur primarily in loose sandy soils due to reduction in volume during or after an earthquake event. The results of the liquefaction analysis indicated several strata were susceptible to liquefaction at the locations explored within the Cove region, but slope stability analysis indicated adequate factor of safety (greater than 1.25) for the short-term conditions in which the residual shear strength the liquefied deposits govern slope stability.

Although the potential for slope stability was not considered to be of significance along the Cove side of the harbor, the consolidation of the liquefiable deposits indicates the potential for settlement and distortions to the seawall. Based upon the empirical procedure described by Tokimatsu and Seed (1987), the post-liquefaction seismically-induced settlement was estimated to be on the order of 1 to 2 inches. The settlement may, therefore, result in distortion to the seawall system.

3.5.6 Earthquake-Induced Flooding

The failure of dams or other water-retaining structures as a result of earthquakes could result in flooding. The potential of earthquake-induced flooding that will affect the site is considered to be low due to the lack of a major dam or water-retaining structure located near the site.

3.5.7 Seiches

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. Because of the partially enclosed configuration of the Dana Point Harbor, the possibility of seiche phenomena occurring within the harbor could be of concern. Further study of wave run-up near the harbor during a major seismic event should be performed during the design phase.

3.5.8 Tsunamis

Tsunamis are waves generated in large bodies of water as a result of change of seafloor topography caused by tectonic displacement. As a result of the proximity of the site to the ocean and its near-sea level elevation, tsunami hazard should be considered during design. McCulloch (1985) predicted a 100-year tsunami event could result in a runup of about 4 feet at the harbor. When combined with high tide, the wave runup may topple the existing seawall. Further study of the potential effect on the seawall should be performed during the design phase by a qualified engineer experienced in coastal engineering.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 Design Considerations

ADA Platforms: The proposed renovation includes the construction of six new pedestrian platforms that comply with ADA specifications at various locations throughout the harbor. The platforms are intended to provide access from the Boardwalk to the gangways that extend to the floating docks. The platforms would typically be situated offshore adjacent to the seawall and the Boardwalk, but the use of this type construction technique would require penetration of the existing submarine concrete panels. Due to their age and to avoid damage to the panels, we understand that the platforms will consist of a structural slab supported by a fixed foundation that will be situated behind (landside) the existing seawall with the slab cantilevering to the gangway, a distance of approximately 8 feet from the harbor-side face of the wall. The foundations for the platform will, therefore, be supported within the soils that underlie the site along the perimeter of the marina. The soils typically consist of existing fill that are then underlain by native alluvial soils and bedrock. The liquefaction potential of the soils and the potential for instability of the slopes will affect the design of the platform foundations if the intent of the design is to maintain serviceability after a seismic event that triggers liquefaction.

Seawall: The results of the liquefaction and post-liquefaction slope stability analyses indicated the potential for slope and seawall instability in the peninsulas adjacent to the Sport Fishing Docks and the Youth & Group facility as well as along the Island. In the event liquefaction occurs, significant lateral displacement is expected to occur of the seawall and surface improvements within the influence of the slope instability. Mitigation measures typically include the in-situ ground modification techniques to reduce the liquefaction potential. Based upon prior discussions with the client and the County of Orange, we understand that liquefaction mitigation is not desired at this time. Recommendations for such improvement techniques are, therefore, not included in this report, but can be provided under separate cover upon request.

Guide Piles: The renovation of the harbor includes reconfiguration the marina, thereby requiring new guide piles to fix the position of the boat docks within the marina. Piles will also be required for permanent slips at the Youth & Group facility, Sport Fishing Docks, and the Embarcadero Docks; and piles will be required for the temporary docks along the western and southwestern sides of the Island as well as the western side of the eastern breakwater.

The pile foundations may consist of either driven pre-cast concrete piles or steel pipe piles, or pre-cast pre-stressed concrete piles set in pre-drilled boreholes socketed into the bedrock. Preliminary project team meetings indicated the preferential alternative is the use of pre-stressed concrete piles set in pre-drilled boreholes to reduce noise nuisance to the existing facility and to the adjacent developments. In addition, a driven pile alternative is expected to encounter driving difficulties due to the in-situ density/consistency of the bedrock and the historical accounts pile driving difficulties during the original construction of the marina. We understand that the existing guide piles consist of a composite section in which the lower region consists of a steel beam "stinger" extending below the bottom of the pre-cast concrete pile.

The termination of several CPT soundings and test borings at depths shallower than planned due to the presumed presence of cobble, boulder and/or rock slab material indicates the potential for difficulties during the installation of pile foundations. Pre-drilling in conjunction with rock coring may be necessary to ensure embedment to the recommended depth.

4.2 ADA Platform Foundations

The primary consideration in the design of the foundations for the ADA Platforms will be the liquefaction potential at the various locations. As previously discussed relative to the stability of the seawall, the subsurface conditions at the borings and CPT soundings along the north (Cove) side of the marina do not suggest extensive damage due to liquefaction based upon the relatively thin thickness and number of soil layers of soils that were potentially susceptible to liquefaction. However, the liquefaction analysis at the majority of the remaining CPT and boring locations indicated a significant potential for liquefaction and a corresponding potential for slope instability and significant lateral displacement. The design of the foundations for the platforms will be dependent upon the desired degree of serviceability in the event of liquefaction.

4.2.1 Spread Footing Foundation

Design Considerations: The platforms will consist of a reinforced structural concrete slab that will cantilever a distance approximately 8 feet beyond the seawall into the marina to provide access to the associated gangways. The platforms will be located adjacent to the seawall and will, therefore, potentially surcharge the wall if the platform is supported at an elevation above the bearing

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grade of the seawall. The platform is, therefore, recommended to be supported at an elevation no shallower than the bearing grade of the seawall. The schematic structural cross-section of the retaining wall indicates the bearing grade is approximately 6 to 6½ feet below current grade. Due to limited lateral/plan dimension of the platform and the eccentric loading associated with the cantilever section of the platform, the most feasible spread footing foundation is considered to be a mass concrete pour to fill the excavation that will be required to establish a consistent bearing grade between the seawall and the platform foundation. For purposes of preliminary analysis presented in this report, the mass concrete pour foundation will be of dimensions that are approximately 7 feet perpendicular to the wall orientation by 10 feet parallel to the wall. The 7-foot dimension will extend from the heel of the existing seawall foundation. The platform slab will cantilever a distance of 11 feet from the edge of the foundation closest to the wall.

Bearing Capacity: The mass concrete foundations may be designed for a maximum net allowable soil bearing pressure of 1,500 psf. The resistance to lateral loads will be derived by the friction developed along the bottom of mass concrete and the passive earth pressure against the sides of the foundation. The sliding resistance may be calculated using a coefficient of sliding friction of 0.30 for foundation concrete placed on the existing soils. The passive earth pressure may be calculated on the basis of an **ultimate** (no safety factor) equivalent passive fluid pressure of 300 psf per foot of foundation embedment. The recommended passive pressure pertains to level grade that extends continuously from the foundation a minimum distance of twice the footing embedment; no passive resistance should be included in design for translation in the direction of the seawall.

The vertical load bearing capacity and the equivalent passive fluid pressure values stated above may be increased by one-third for design under short-term, transient loading conditions.

The footing excavations are recommended to be reviewed and approved by a representative of the geotechnical engineer at the time of construction to verify that the footings are supported in suitable bearing soils. Due to the presence of fill and variances in support characteristics, some additional excavation may be necessary based upon the conditions encountered in the field. The actual depth of overexcavation should be reviewed by the geotechnical engineer prior to concrete placement after overexcavation to the recommended depth.

Upon completion of subgrade evaluation prior to construction and design as recommended in this report, the settlement of the platform foundations is estimated to be less than 1 inch. Care should be used in the structural detailing of the slab cantilever to reduce the vertical load that will be transferred to the and thereby supported by the seawall foundation.

4.2.2 Drilled Pier Foundations

Design Considerations: A drilled pier foundation system is considered to be a feasible alternate foundation system for support of the platforms. A drilled pier foundation that is extended through the existing fill and the potentially liquefiable soils to bear within the bedrock will reduce overexcavation as required for a shallow spread footing foundation system and can be designed to reduce the effects of liquefaction and maintain serviceability.

The slope stability analyses indicate the potential for instability and lateral translation. The potential slip surfaces typically extend from the toe of the submarine slope and intersect the ground surface at distances of 12 and 17 feet behind the seawall at the analyzed cross-sections, Island West at CPT-3 and Sport Fishing Docks at CPT-10. The positioning of the platforms and the supporting foundations within the potential slide mass requires the design of the pier foundations to include the associated lateral load due to soil translation. Based upon the encountered subsurface profiles at the boring and CPT sounding locations, the design of the pier foundations to support the platforms may be classified in three categories:

- The platforms along the northern shore of the marina (Cove side) where the liquefaction potential was found to be relatively minor and did not present a significant potential for slope instability;
- The platforms located in the peninsula area adjacent to the Sport Fishing docks in the eastern region and the Youth and Group facility in the western region where a relatively tick and continuous zones of potentially liquefiable soils were identified; and
- The platforms along the northern shore of the Island where the potential for liquefaction and slope instability was also identified, but the effect on pier design was found to be somewhat less than the peninsula areas.

Lateral Load Conditions: In the area where a significant risk of liquefaction induced slope instability exists, the design of the drilled pier foundations is anticipated to be controlled by the resistance to lateral load relative to the axial compressive load to support the platform structures. The load applied to the piers due to liquefaction induced slope instability is dependent upon the depth at which the critical slip surface intersects the pier and the required resisting force to be provided by the pier foundations to obtain a suitable factor-of-safety. The results of the slope stability analysis indicates the piers must provide an equivalent load of 1.5 kips per lineal foot (klf) for the platforms located on the Island; and a load of 8.7 klf for the platforms located on the peninsulas. On the basis of this analysis, it is recommended that the pier foundations that support the platforms on the Island be designed for a lateral load surcharge calculated on the basis of an equivalent fluid pressure of 30 pcf per foot of pier diameter. The pier foundations that support the platform in the peninsula areas are recommended to be designed for an equivalent fluid pressure of 105 pcf per foot of diameter. The equivalent fluid pressures attributed to soil surcharge should be applied from the top of the pier to a depth of 15 feet below grade for the locations on the Island; and to depth of 13 feet for the locations on the peninsulas. The total lateral load applied to the piers should be determined on the basis of an area of influence behind the pier equal to twice the diameter.

The resistance to the lateral load will be provided by the passive earth pressure that develops along the lengths of the piers that extend below the critical slip surface. The encountered subsurface profiles generally indicate the presence of soils to a depth of 10 feet below the critical slip surface followed by the formational bedrock. In some locations, portions of these soils are susceptible to liquefaction and, therefore, provide significantly lower resistance. On this basis, the **ultimate** (no safety factor) equivalent passive fluid pressure for use in design is recommended to be 130 pcf for the portion of the piers extending below the critical slip surface a distance of 10 feet to the bedrock contact. The **ultimate** equivalent passive fluid pressure is recommended to be 450 pcf for the portion of the piers that are founded within the formational bedrock material.

The previously stated values for passive equivalent fluid pressure are based upon **ultimate** strength capacity; the appropriate factor of safety should be applied by the structural engineer.

Axial Compressive and Tensile Loads: The pier foundations are recommended to be extended to bear within the formational bedrock that underlies the harbor. The axial compressive capacity of the drilled pier foundations is recommended to be calculated on the basis of a net allowable bearing pressure of 6,000 pounds per square foot (psf) for piers extended to bear a minimum depth into the bedrock the equivalent of two pier diameters. Piers that are extended to bear at greater depths below the top of the bedrock stratum may include the additional load bearing capacity developed by skin friction. In this event, the allowable axial compressive capacity may be increased by 1,100 psf per foot of pier diameter per foot of embedment below a depth of 2 pier diameters into the bedrock.

The resistance to uplift may be calculated on the basis of the weight of the pier and the friction that develops along the surface. The unit skin friction that is recommended for use in calculation of uplift resistance is 600 psf per foot of diameter per foot of embedment into the bedrock.

Construction Considerations: The drilling operations associated with drilled pier construction are recommended to be observed and evaluated by a representative of the geotechnical engineer to allow further evaluation of the actual subsurface conditions and verify proper embedment depth into the bedrock material. If the end-bearing capacity of the piers is required based upon structural design, the drilling operations are recommended to include the use of clean-out bucket to remove loose/sloughed soils that accumulate in the bottom of the borehole. However, based on the encountered soil conditions and shallow groundwater conditions, it is anticipated that the construction of drilled piers will require drilling mud and/or temporary casing to prevent caving. Furthermore, the placement of concrete is recommended to be performed by tremie to displace groundwater and drilling fluid. Care should be used in the installation/removal of temporary casing and/or the use of slurry for borehole stability to reduce the potential for adversely affecting the frictional resistance of the soils and thereby reduce the load capacity of the piles. A minimum concrete head of 5 feet is recommended to be maintained at all times during the removal of the temporary casing to prevent caving.

Drilling for the pier foundations may encounter difficulties due to occasional oversize and/or very dense material within the fill and the underlying native soils. Drilling difficulties are expected to be encountered in the areas of the Youth and Group facility and in the eastern region of the Island where our test borings and/or

CPT soundings required termination at depths shallower than planned due to refusal. Borehole drilling may, therefore, require special techniques such as rock coring or other methods to extend through zones of resistance to achieve design depth.

4.3 Seawall Stability

The structural integrity of the seawall system was previously evaluated on a preliminary basis by Bluewater Design Group (Bluewater 2003). As part of their evaluation, the static stability of the retaining wall was calculated using geotechnical parameters presented in a report prepared by our firm for the preliminary evaluation of the Dana Point Master Plan (Leighton 2002). The results of the previous stability analysis, using parameters presented for future development proposed for the marina, indicated a factor of safety of 1.0 with respect to sliding and 4.0 with respect to overturning. Recommendations are subsequently presented in this report that may be used to refine the stability analysis of the retaining wall based upon conditions encountered in close proximity to the retaining walls that are expected to be more representative of actual conditions.

The results of the current study indicate the seawall along the northern boundary (Cove region) of the harbor may be considered to be generally stable with respect to the overall rotational or gross stability in spite of the presence of layers of soils that exhibited the susceptibility to liquefaction. The occurrence of liquefaction is, however, expected to result in some post-liquefaction settlement due to consolidation of the liquefied soils. The magnitude of this settlement was estimated to be on the order of 1 to 2 inches using currently available empirical prediction techniques. The potential for liquefaction was considered to be of greater significance along the southern seawall (Island region) where the potential for liquefaction to occur is expected to result in slope instability and lateral displacements.

The following recommendations for lateral pressure, foundation bearing and seismic loading conditions are based upon the conditions encountered at the test boring locations. The design values for earth pressure presented herein do not contain an appreciable factor of safety; the structural engineer should apply the applicable factors of safety and/or load factors for use in design evaluation. Due to the presence of fill, significant variance may exist between the boring locations in areas that have not been explored. As a result, additional exploration of the seawall areas may be warranted to verify the design parameters subsequently presented.

4.3.1 Lateral Earth Pressure

Static Conditions: The retaining wall of the seawall system is considered to be a cantilever structure. As such, the Active earth pressure condition is considered to be appropriate for use in the evaluation of the stability. The evaluation of the wall stability may be based upon earth pressure modeled as a fluid with equivalent fluid weight of 37 pcf for the Active earth pressure condition. The recommended earth pressure condition pertains to drained conditions behind the wall. In consideration of the probable long term water condition behind the wall corresponding to a depth of approximately 5 feet below grade, i.e., approximate El. 0 feet MLLW, the water that may accumulate behind the wall will be due to infiltration from the ground surface. However, based upon our understanding of wall design which includes joints between adjacent stemwall panels, the potential for the accumulation of water and development of hydrostatic pressures is considered to be relatively low.

Seismic Conditions: The magnitude of the surcharge load subjected to the retaining wall of the seawall system will be dependent upon the magnitude of the peak ground acceleration experienced at the site. The seismic load surcharge is typically modeled as a concentrated load situated at a distance above the base of the wall equivalent to approximately 60 percent of the wall height (0.6H). The seismic load surcharge is recommended to be of 0.35 kips for seawalls with a height of 6 feet 3 inches and a equivalent horizontal ground acceleration of 0.19g, which is approximately equivalent to 50 percent of the PHGA. The magnitude of the seismic increment load for walls of other height or other seismic scenarios can be determined on a case-by-case basis.

Static Surcharges: In addition to the above lateral pressures from retained earth, lateral pressures from other superimposed loads, such as those from automobile traffic and adjacent structures should be added to the load imposed upon the wall, if the surcharge is located a distance from the back of the wall equal to or less than the height of the wall. The magnitude of the surcharge load depends upon the size of the surface area that is subjected to a vertical load relative to the wall height and distance form the wall.

The lateral surcharge may modeled as a uniform pressure distribution with a pressure intensity equivalent 31 percent of the vertical surcharge for loaded areas that are adjacent to the wall and of large lateral extent. The surcharge attributed to surface loads of limited lateral extent and/or situated at various distances from the wall requires analysis on a case-by-case basis.

4.3.2 Lateral Load Resistance

The soil resistance available to withstand lateral loads is a function of the frictional resistance along the base of the foundation and the passive resistance that may develop as the face of the structure tends to move into the soil. The frictional resistance between the base of the retaining wall foundations and the subgrade soil may be computed using a coefficient of friction of 0.39. Based upon the design of the wall in which the toe of the retaining wall is not in contact with the concrete panels of the revetment and the fact that the wall is situated at the crest of the slopes that descend into the basins, the evaluation of wall stability should include only frictional resistance along the base of the foundation and no contribution from passive resistance.

4.3.3 Bearing Capacity

The retaining wall foundation is situated at the crest of the slope that descends into the adjacent basin with no appreciable footing embedment. The lack of embedment and proximity to the slope affects the allowable soil bearing pressure that may be used in the evaluation of the bearing capacity. Based upon the near surface soils conditions encountered at our test boring locations to depth of significant foundation influence and the reported footing width of 6 feet – 3 inches, a maximum net allowable soil bearing pressure of 1,800 psf is recommended for use in evaluation of bearing capacity under static conditions. The recommended bearing capacity may be increased to 2,400 psf for short-term loading conditions.

4.4 Seismic Design Parameters

Based upon the California Building Code (CBC, 1998), the site is within Seismic Zone 4 with a Z factor of 0.4, as is the case for most of Southern California. A soil profile type of S_D (i.e., stiff soil profile) should be used, as shown in Table 16-J of the 2001 CBC.

Seismic design may be based on a Seismic Source Type of "B" (i.e., Newport-Inglewood Fault located approximately 3.5 km from the site) with Near-Source Factors N_a and N_v of 1.15 and 1.4, respectively.

4.5 Guide Pile Recommendations

Design Considerations: The guide piles that will be installed within the marina will be primarily subjected to lateral loading conditions associated with minor wave action, wind and more significantly by the impact loads associated from boats that dock at the platforms. The magnitude of the lateral loads and the tolerable pile deflections have not yet been provided, but the loads are anticipated to be relatively low while the allowable lateral deflections are expected to be greater than deflections that are typically used for design of buildings. For preliminary design purposes, we have assumed that deflections of ¼, ½ and 1 inch at the mudline will result in total deflection at the top of the piles that will be within tolerable levels.

Based upon the conditions encountered at the CPT soundings and test borings located onshore, the submarine profile is anticipated to be such that the thickness of seafloor sediments overlying the bedrock increases progressing across the harbor from the Cove to Island sides. Consequently, the load capacity of the guide piles will be influenced by the lower strength characteristics of the seafloor sediments in the southern region of the harbor and in the entry channel along the south side of the Island. The geophysical seafloor profiling will provide additional information regarding the subsurface conditions in the areas of pile installations. Upon receipt and review of the profiling report, an addendum to this report will be issued that presents further recommendations and construction considerations for pile construction.

The results of the seawall study indicated the potential for liquefaction along the southern seawall (Island region) which was expected to result in slope instability and lateral displacements. The slope movements that may occur as a result of liquefaction could impart significant additional lateral load on the guide piles within the zone of slide movement. The loads associated with slope movement may require evaluation and consideration in the design of the piles depending upon the distance between the piles and the seawall.

Lateral Load Capacity: Evaluation of lateral load capacity has been conducted on the basis of the anticipated submarine profiles at the locations of CPT-1/Boring-1 (Cove side west) and CPT-3/Boring-3 (Island side west) using the software package *LPile Plus* by

Ensoft. The subsurface profiles used in these analyses are considered to be representative of the harbor and of the areas in which the temporary docks will be installed. A summary of the analysis is included in Appendix F. The analysis was conducted for either a 14-inch square or a 16-inch diameter precast concrete pile extended to depths of 10, 15 and 20 feet into the bedrock.

The selection of appropriate pile type and construction technique will be somewhat influenced by the required embedment depth to achieve the required lateral load capacity and the associated lateral deflection at the top of the pile. The results of the analysis depict the deflection of the respective piles and the distribution of shear forces and bending moment along the length of the pile. The "depth" indicated as "0 feet" represents the mudline which was assumed to be approximately El. -10 feet MSL at the shore along the Cove side and at the location of Temporary Dock T-5; El. -18 feet MSL along the shore of the Island; and El. -27 to -28 feet MSL in the areas of Temporary Docks T-1 through T-4 assuming that El. 0 feet MSL corresponds to approximately 10 feet below the ground surface of the Cove and Island.

The results of the preliminary analysis indicate that pile embedment will be approximately 10 to 15 feet into the underlying bedrock with a pile tip elevation ranging from El. -20 to -25 in the northern region of the West Basin as well as the northern and southern regions of the East Basin; El. -28 to El. -33 in the southern region of the West Basin; and El. -38 to -43 at Temporary Docks T-1 through T-4. Upon completion of the geophysical seafloor profiling, revision to the pile embedment depths may be warranted.

Construction Considerations: Lateral loading is expected to be the primary load demand of the piles. Consequently, the piles should be in continuous contact with the adjacent soils and bedrock to provide lateral load resistance. Therefore, the selection of pile type and method of installation will be highly dependent upon this primary consideration for design. The use of driven piles is expected to result in piles properly embedded within the adjacent materials such that continuous contact with undisturbed material is maintained. However, pile driving will result on noise and vibrations that could be a disturbance to marina users, occupants of the harbor facilities and the local residents.

The preferred method of pile installation is understood to be piles that are drilled and set in-place within pre-drilled boreholes. In-situ construction techniques will minimize disturbance and yet allow proper continuity between the piles and boreholes to achieve lateral load resistance. Therefore, we anticipate that the piles will be set in a borehole of slightly greater dimension in which the pile is secured by grout injection around the perimeter of the pile, filling the annular space. Alternate construction techniques such as sand-jetting may be feasible, but field testing will be warranted to verify proper contact between the pile and borehole and verify lateral load capacity.

The anticipated depth of embedment relative to the consistency/hardness of the bedrock material indicates special considerations exist for the installation of driven piles. At a minimum, precast concrete piles are expected to require a driving shoe integrally fabricated with the pile. Alternatively, the piles may consist of a composite section in which the lower portion consists of steel H-beam protrusion ("stinger") to serve as a driving point to break the bedrock. Based upon the driving resistance encountered during geotechnical sampling, a minimum driving energy of 150 ft-kip is anticipated to be necessary for pile driving. However, a specific evaluation of the hammer-cushion-pile system that is planned for use at the site can be performed once potential combinations have been identified to determine the feasibility of pile driving. Of primary importance in pile driving is to maintain driving stresses within specific ranges of the compressive and tensile strength of the pile material. Guidelines published by the Federal Highway Administration (FHWA) indicates the maximum driving stresses for conventionally reinforced concrete piles is $0.85 f'_c$ in compression and $0.70 f_y$ in tension.

An alternate pile type that may be considered consists of steel pipe piles or steel H-beams (H-piles) fitted with a driving shoe to allow penetration of the bedrock. The use of steel pipe or beam piles within a marine environment will, however, require provisions to protect the steel wall of the pipe or steel of the beam from corrosion. The FHWA guidelines indicate the maximum driving stresses for steel H-piles and steel pipe piles is less than $0.9 f_y$.

The use of pre-drilled boreholes to facilitate pile driving may be feasible, but the performance of submarine drilling will present difficulties with borehole stability where the seafloor sediments are of significant thickness. Pre-drilling is recommended to be performed such that the borehole diameter is no larger than the diameter of a circular pile or the width of a square pile so that once driven to the design tip elevation, sufficient continuity exists between the pile and the adjacent soils and bedrock.

4.6 Corrosivity Considerations

Based on the results of soluble sulfate content testing, we recommend that foundation concrete for the proposed structures at the subject site be designed in accordance with the "moderate" category of the concrete mix design guidelines contained within the California Building Code for resistance to sulfate exposure.

The results of the resistivity and chloride content tests of the soil indicate that these soils are severely corrosive to metals. Significant precautions are, therefore, anticipated to be necessary relative to corrosion. Consideration should be given to retaining a corrosion engineer to obtain recommendations for the protection of metal components embedded in the site soils.

5.0 LIMITATIONS

The conclusions and recommendations presented in this report have been based upon the generally accepted principles and practices of geotechnical engineering utilized by other competent engineers at this time and place. No other warranty is either expressed or implied.

The conclusions and recommendations presented in this report have been based upon the subsurface conditions encountered at discrete and widely spaced locations and at specific intervals below the ground surface. Due to the inherent variance in soils conditions, variability may be encountered during construction. Where encountered during construction, such variances should be brought to our attention to determine the impact upon the recommendations presented in this report.

This report has been prepared for the use of our client for the project described in this report. The report may not be used by others without the written consent of our client and our firm.

Important Information About Your
Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

The following information is provided to help you manage your risks.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you ASFE-member geotechnical engineer for more information.

ASFE
THE BEST PEOPLE ON EARTH

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e-mail: info@asfe.org www.asfe.org

PROJECT MEMORANDUM

To: URS / Cash and Associates
5772 Bolsa Avenue, Suite 100
Huntington Beach, California 92649

Date: May 11, 2007

Project No. 600024-003

Attention: Mr. Randy Mason

From: John E. Haertle, G.E. 2352

Subject: Preliminary Recommendations for Guide Piles, Dana Point Harbor Renovation, Dana Point, California

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In accordance with your request, Leighton Consulting, Inc. (Leighton) presents this project memorandum to address the selection, design and construction of the guide piles within the marina as part of harbor renovation. This memorandum is presented as an addendum to the Draft Geotechnical Report (March 13, 2007) prepared by our firm and submitted for review. The focus of the geotechnical report was a preliminary evaluation of the seawall of the Dana Point Harbor. As part of the preliminary evaluation, subsurface exploration was performed in close proximity to the existing seawall at predetermined locations.

Field Exploration

Four (4) soil borings were drilled to depths that ranged from 34 to 50 feet below the current grade by a truck-mounted drilling rig using rotary drilling techniques. All borings were extended into the formational bedrock that underlies the area. In addition to the soil borings, five (5) CPT soundings were performed in close proximity to the boring locations so that samples collected from the test boring locations could be correlated to the CPT data and the profile penetrated by the CPT probe. The CPT soundings met with refusal at depths of 12½ to 33 feet below grade. Based upon the boring data, all CPT soundings except for CPT-4 were terminated in the formational bedrock while CPT-4 was presumed to have been terminated on cobble or boulder material. The locations of the soil borings and CPT soundings are shown on Figure 2, Boring and CPT Exploration Map included in the referenced report.

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Subsurface Soil Conditions

The results of the field exploration indicated the presence of fill to depths that varied from approximately 13 feet on the Cove side of the harbor to depths of approximately 23 to 28 feet below the Island along the southern side of the harbor. The fill that underlies the Cove side of the harbor typically consisted of fine to medium grained sands with varying clay content that exhibited loose to medium dense relative density on the basis of field testing (N-values). The fill material encountered below the Island side of the harbor also consisted primarily of sand with greater silt and occasional clay content. The fill generally exhibited loose to medium dense relative density on the basis of field testing. Field tests that indicated dense relative density are not considered to be indicative of the actual relative density of the material but rather the influence of the presence of oversize (cobble and boulder) material within the fill.

The fill was underlain by native soils comprised of loose relative density sands with varying clay content to a depth of about 16 feet where bedrock was encountered on the Cove side of the marina. The bedrock was encountered at depths of 23 to 28 feet below grade below the Island, but native soils were not identified in the samples recovered of the material that overlain the bedrock.

Bedrock of the Capistrano formation was encountered below the fill and native soils at the depths described above. The bedrock typically consisted of siltstone, but a layer of sandstone was encountered at the location of Boring B-3 in the western region of the Island that overlies the siltstone to a depth of 38 feet.

Preliminary Conclusions and Recommendations

Design Considerations: The guide piles that will be installed within the marina will be primarily subjected to lateral loading conditions associated with minor wave action, and more significantly by the impact loads associated from boats that dock at the platforms. The magnitude of the lateral loads and the tolerable pile deflections have not yet been provided, but the loads are anticipated to be relatively low while the allowable lateral deflections are expected to be greater than deflections that are typically used for design of buildings. For preliminary design purposes, we have assumed that deflections of ¼, ½ and 1 inch at the mudline will result in total deflection at the top of the piles that will be within tolerable levels.

Based upon the conditions encountered at the CPT soundings and test borings located onshore, the submarine profile is anticipated to be such that the thickness of seafloor sediments overlying the bedrock increases progressing across the harbor from the Cove to Island sides. Consequently, the load capacity of the guide piles will be influenced by the lower strength characteristics of the seafloor sediments in the southern region of the harbor.

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The results of the seawall study indicated the potential for liquefaction along the southern seawall (Island region) which was expected to result in severe slope instability and large lateral displacements. The slope movements that may occur as a result of liquefaction could impart significant additional lateral load on the guide piles within the zone of slide movement. The loads associated with slope movement will require evaluation and consideration in the design of the piles.

Lateral Load Capacity: A preliminary evaluation of lateral load capacity has been conducted on the basis of the anticipated submarine profiles at the locations of CPT-1/Boring-1 (Cove side west) and CPT-3/Boring-3 (Island side west) using the software package *LPile Plus* by Ensoft. A summary of the analysis is included in the attached figures. The analysis was conducted for either a 14-inch square or a 16-inch diameter precast concrete pile extended to depths of 10, 15 and 20 feet into the bedrock.

The selection of appropriate pile type and construction technique will be somewhat influenced by the required embedment depth to achieve the required lateral load capacity and the associated lateral deflection at the top of the pile. The results of the analysis depict the deflection of the respective piles and the distribution of shear forces and bending moment along the length of the pile. The "depth" indicated as "0 feet" represents the mudline which was assumed to be approximately El. -10 feet MLLW at the shore along the Cove side and El. -18 feet MLLW along the shore of the Island side assuming that El. 0 feet MLLW corresponds to approximately 10 feet below the ground surface of the Cove and Island.

The results of the preliminary analysis indicate that pile embedment will be approximately 10 to 15 feet into the underlying bedrock with a pile tip elevation ranging from El -25 to El. -28 from the Cove to the Island regions of the marina.

Construction Considerations: Lateral loading is expected to be the primary load demand of the piles. Consequently, the piles should be in continuous contact with the adjacent soils and bedrock to provide lateral load resistance. Therefore, the selection of pile type and method of installation will be highly dependent upon this primary consideration for design. The use of driven piles is expected to result in piles properly embedded within the adjacent materials such that continuous contact with undisturbed material is maintained.

The anticipated depth of embedment relative to the consistency/hardness of the bedrock material indicates special considerations exist for the installation of driven piles. At a minimum, precast concrete piles are expected to require a driving shoe integrally fabricated with the pile. Alternatively, the piles may consist of a composite section in which the lower 3 to 5 feet consist of steel beam protrusion ("stinger") to serve as a driving point to break the bedrock.

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An alternate pile type that may be considered consists of steel pipe piles or steel beams (H-piles) fitted with a driving shoe to allow penetration of the bedrock. The use of steel pipe or beam piles within a marine environment will, however, require provisions to protect the steel wall of the pipe or steel of the beam from corrosion.

The use of pre-drilled boreholes to facilitate pile driving may be feasible, but the performance of submarine drilling will present difficulties with borehole stability where the seafloor sediments are of significant thickness. Pre-drilling is recommended to be performed such that the borehole diameter is no larger than the diameter of a circular pile or the width of a square pile so that once driven to the design tip elevation, sufficient continuity exists between the pile and the adjacent soils and bedrock. Alternate construction techniques may be feasible depending upon the experience of the contractor, but continuity must be maintained to ensure lateral load resistance within the range of tolerable lateral deflection.

Pile installation by other methods that involve drilling and cast-in-place techniques may also be feasible, but the primary consideration will be borehole stability within the seafloor sediments. On a conceptual basis, one scenario may consist of driving a steel casing a sufficient depth below the soil/bedrock contact to prevent caving with the remainder of the pile constructed by drilling to the design depth followed by installation of reinforcing bars and casting concrete.

Closing

The conclusions and recommendations presented in this memorandum are considered to be preliminary in nature based upon the available subsurface data obtained from our recent and prior exploration of the harbor facility. As discussed in this memorandum, the design and installation of the guide piles include considerations regarding the varying depth of seafloor sediments and the presence of formational bedrock at relatively shallow depth below the harbor bottom. Techniques exist that can provide a more detailed depiction of the seafloor bottom and, in particular, the contact between the sediments and bedrock which could be of benefit in determining pile length and potentially driving difficulties.

We recommend that the information presented herein and the subsurface boring and CPT logs of the Draft Geotechnical Report be reviewed with prospective piling contractors to better evaluate the most feasible manner of guide pile construction. Further study of the harbor bottom and borings and CPT soundings conducted in other areas of interest can also be of benefit in refining pile design and construction.

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Lateral Load Capacity

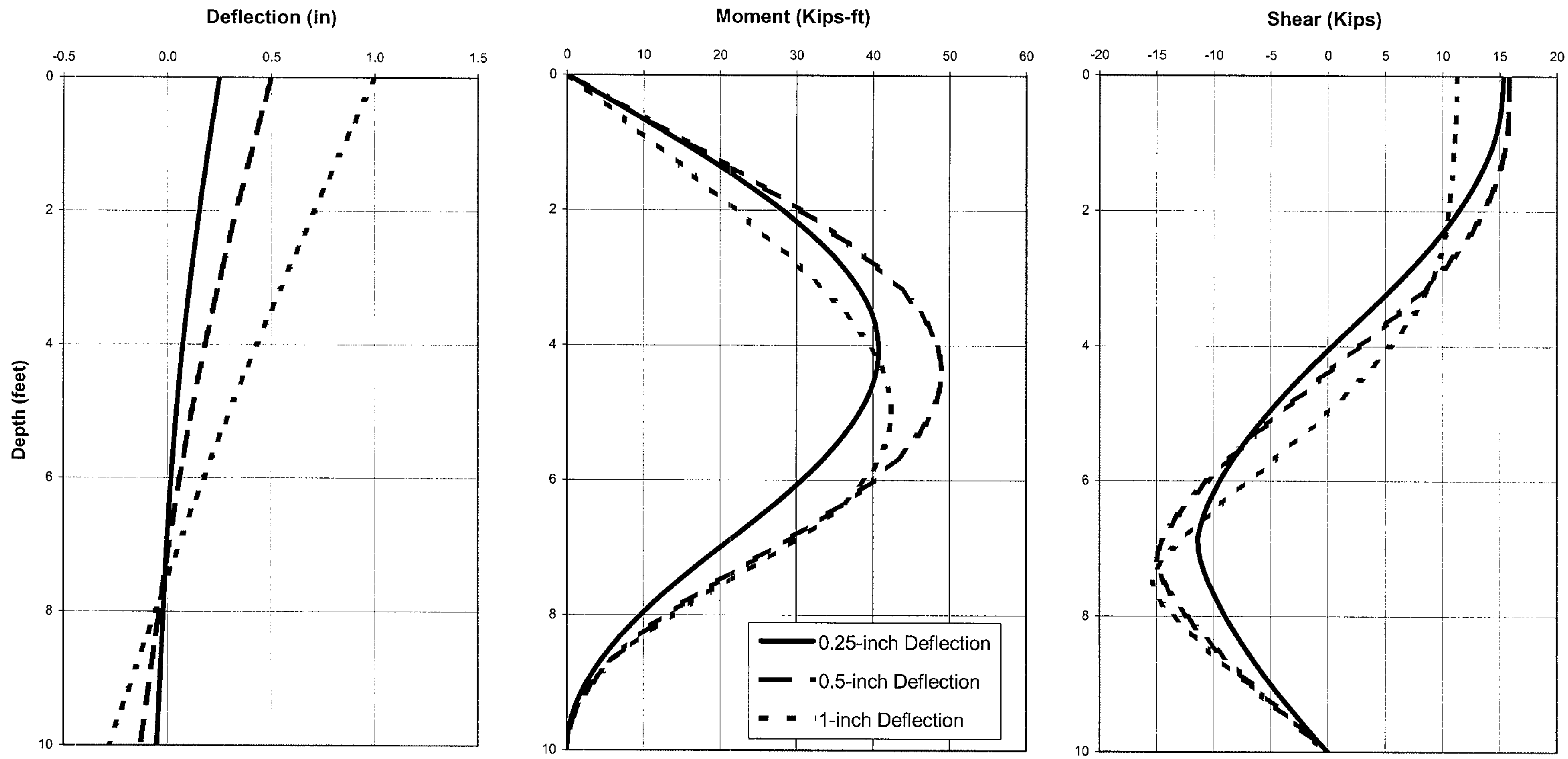
14-inch Wide Precast Concrete Square Piles

Cove West

Pile Tip Embedment at 10, 15 and 20 feet
Into the Bedrock

Graphical Summary of Pile Deflection, Shear and
Bending Moment Distribution along Pile Length

SECTION A-A' (Boring B-1)



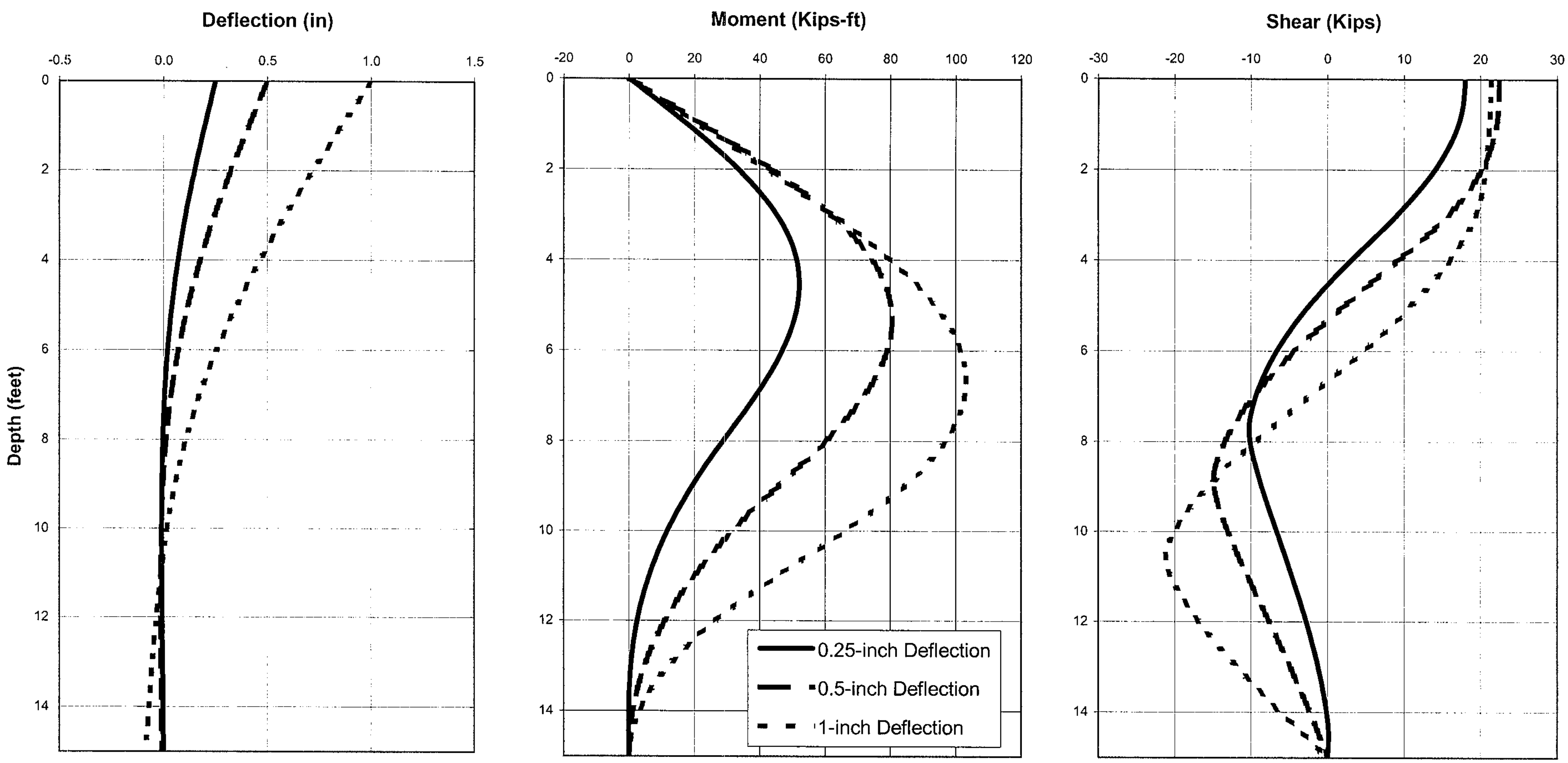
NOTE: Depth is depth of embedment below harbor bottom (k500pci)

LATERAL LOAD CAPACITIES: 14-INCH SQUARE CONCRETE PILE
10-FOOT EMBEDMENT INTO BEDROCK
PRELIMINARY SEAWALL EVALUATION
DANA POINT HARBOR

Project Name:	Dana Point Harbor
Project No.:	600024-003
Date:	4/23/07

Figure 1a

SECTION A-A' (Boring B-1)



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15-FOOT EMBEDMENT INTO BEDROCK
PRELIMINARY SEAWALL EVALUATION
DANA POINT HARBOR

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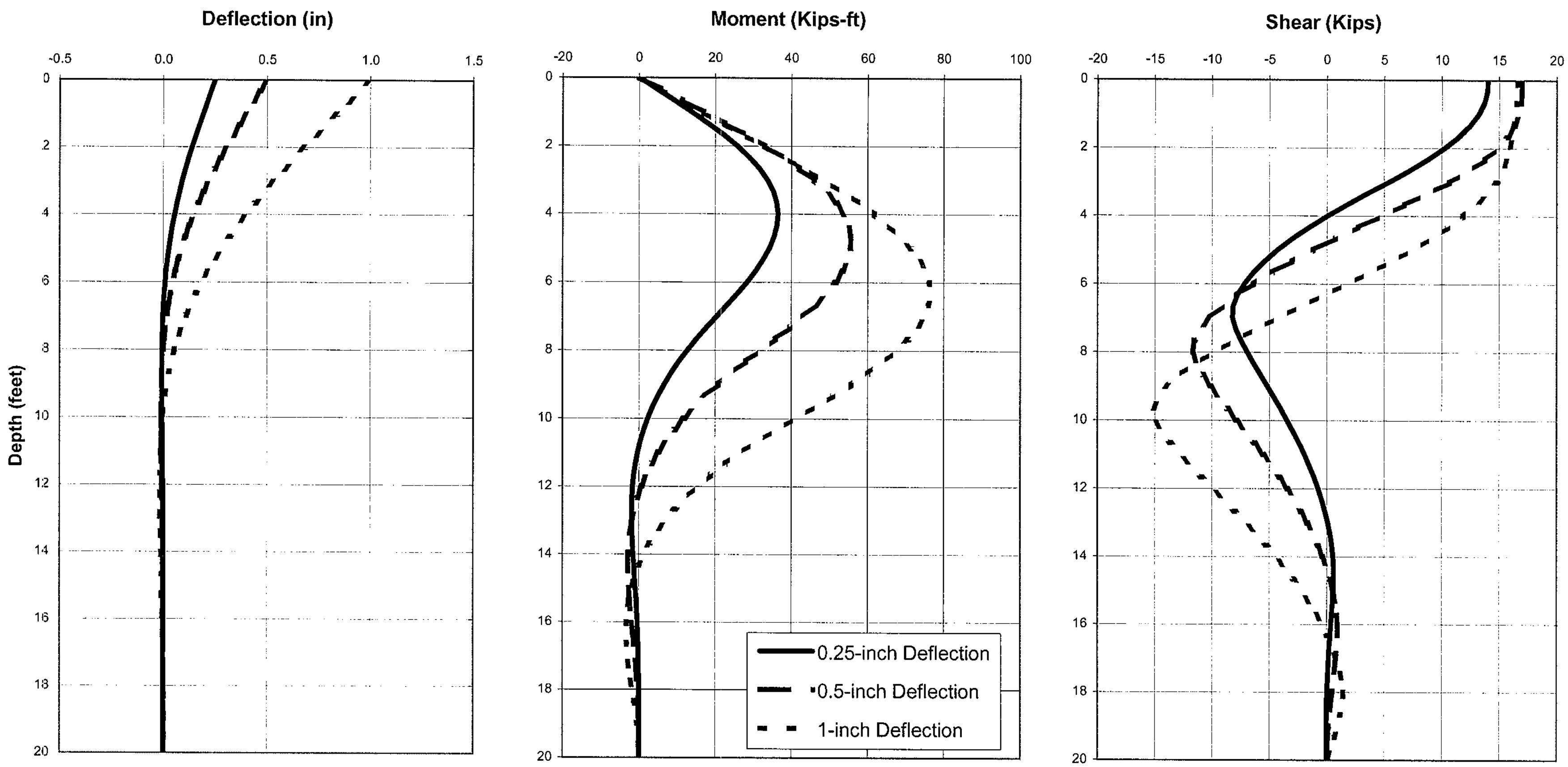
Figure 1b

SECTION A-A' (Boring B-1)



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PRELIMINARY SEAWALL EVALUATION
DANA POINT HARBOR

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Figure 1c

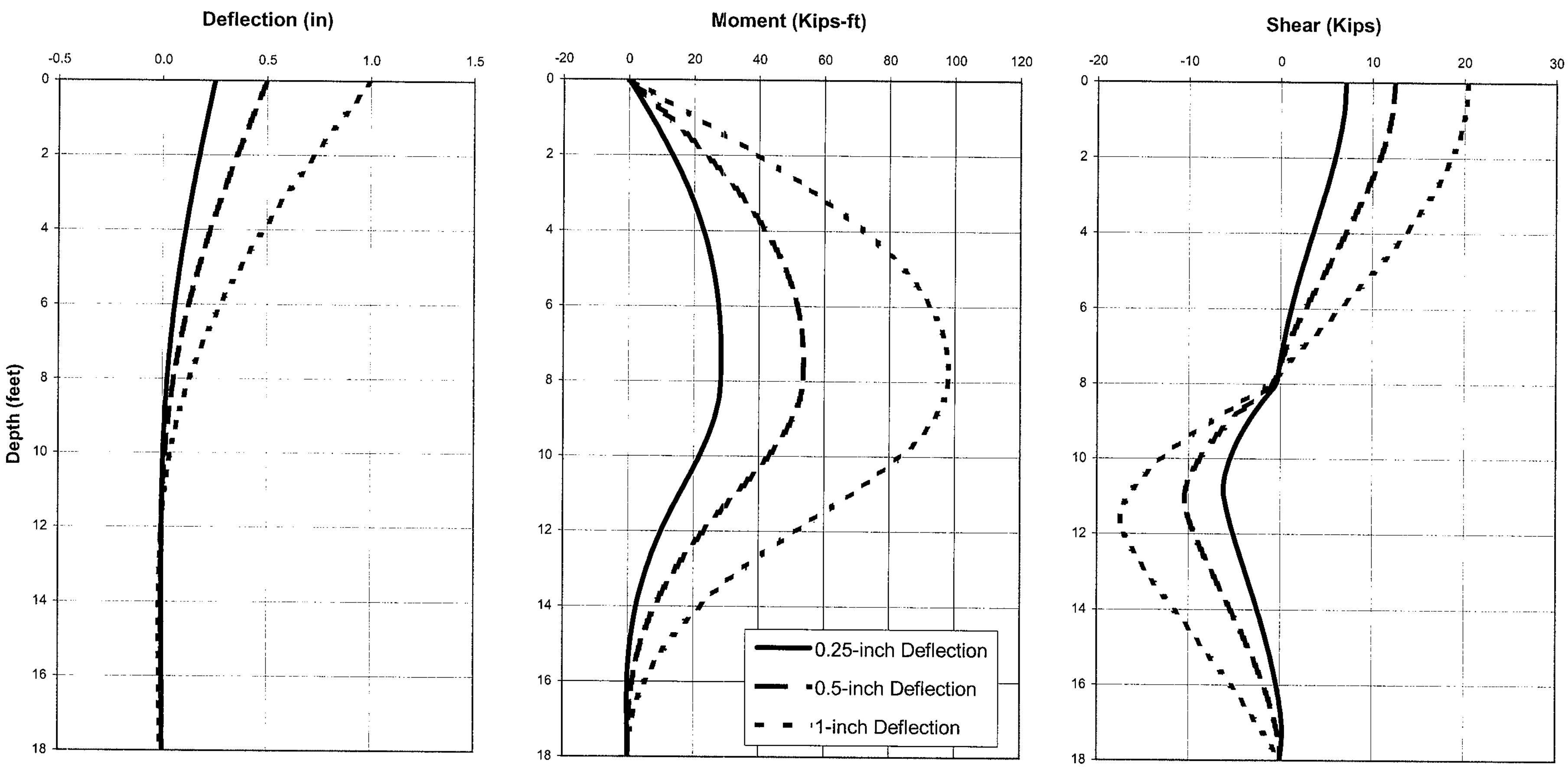
14-inch Wide Precast Concrete Square Piles

Island West

Pile Tip Embedment at 10, 15 and 20 feet
Into the Bedrock

Graphical Summary of Pile Deflection, Shear and Bending Moment Distribution along Pile Length

SECTION B-B' (Boring B-3)



NOTE: Depth is depth of embedment below harbor bottom (k500pci)

LATERAL LOAD CAPACITIES: 14-INCH SQUARE CONCRETE PILE
10-FOOT EMBEDMENT INTO BEDROCK
PRELIMINARY SEAWALL EVALUATION
DANA POINT HARBOR

Project Name:	Dana Point Harbor
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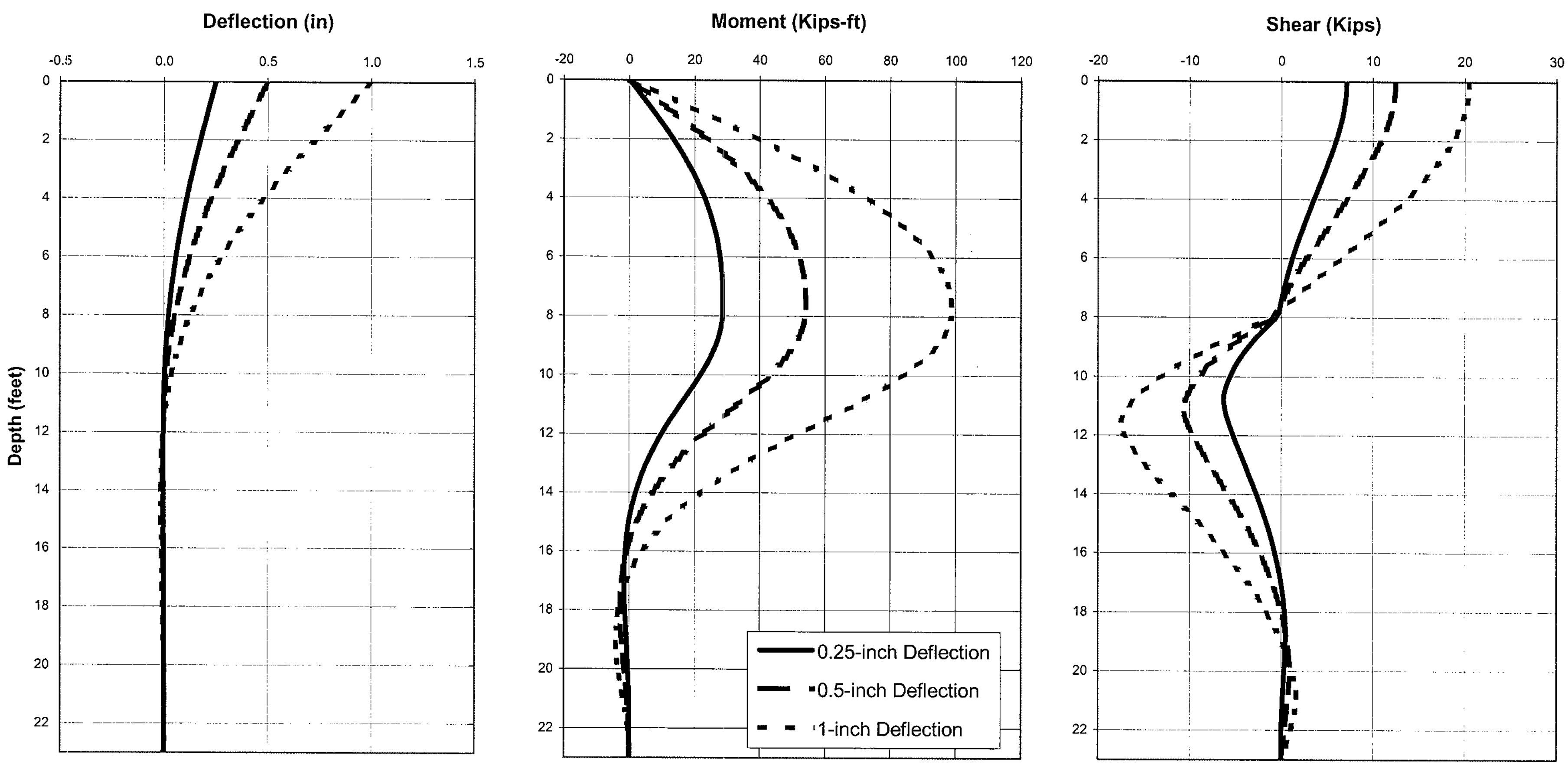
Figure 2b

SECTION B-B' (Boring B-3)



NOTE: Depth is depth of embedment below harbor bottom (k500pci)

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15-FOOT EMBEDMENT INTO BEDROCK
PRELIMINARY SEAWALL EVALUATION
DANA POINT HARBOR

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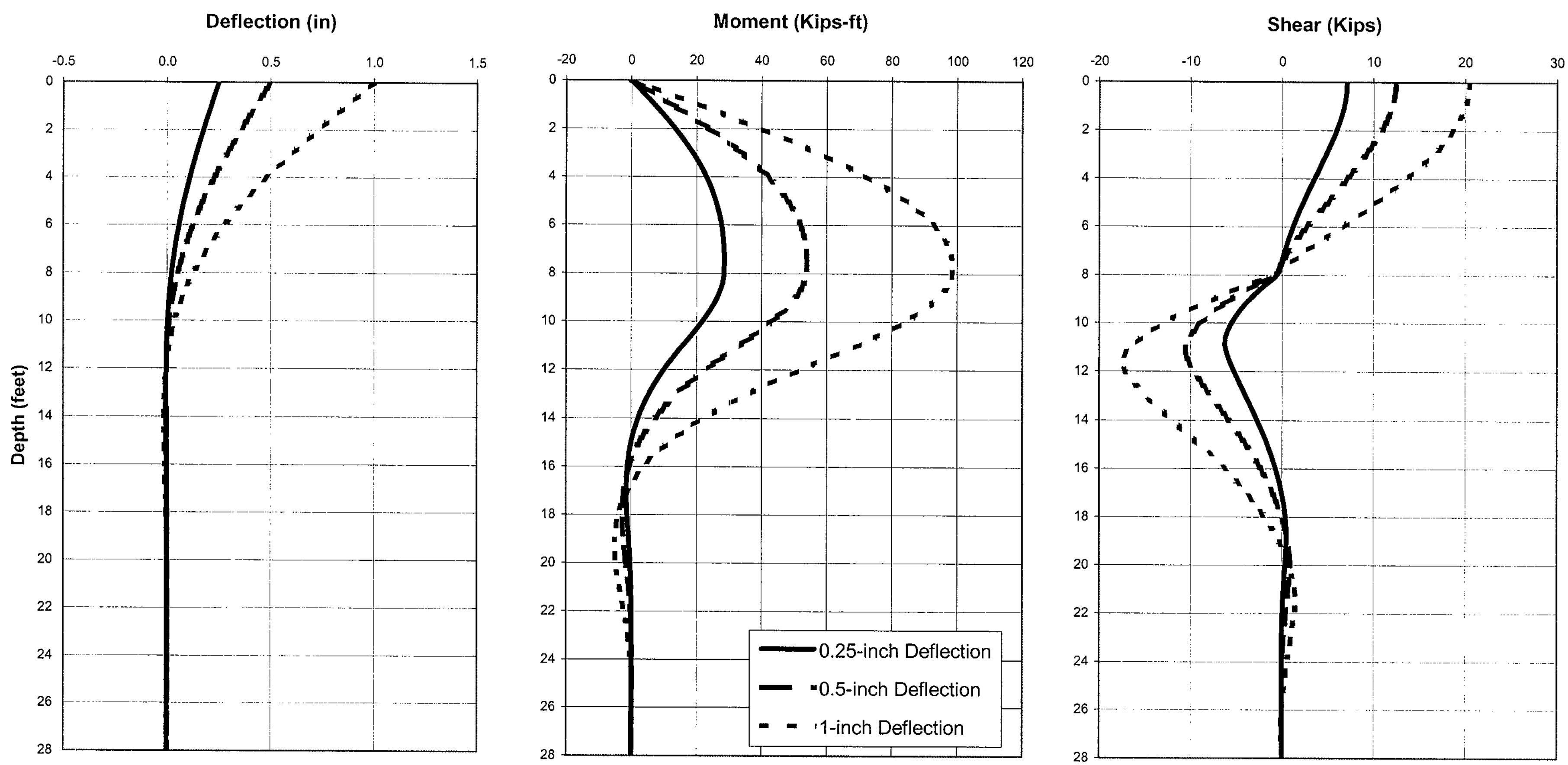
Figure 2c

SECTION B-B' (Boring B-3)



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20-FOOT EMBEDMENT INTO BEDROCK
PRELIMINARY SEAWALL EVALUATION
DANA POINT HARBOR

Project Name: Dana Point Harbor
Project No.: 600024-003
Date: 4/23/07

Figure 3d

Lateral Load Capacity

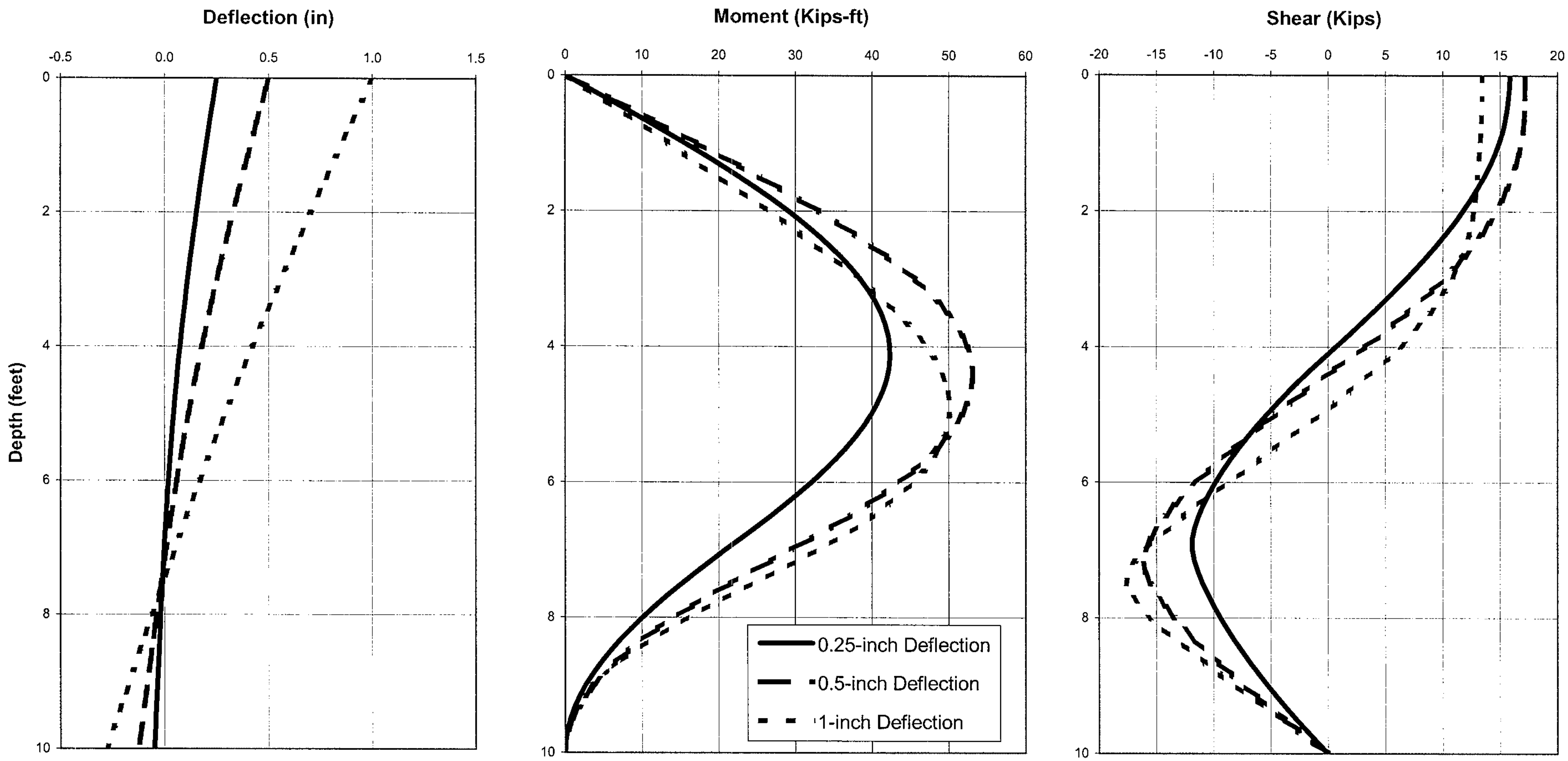
16-inch Diameter Precast Concrete Circular Piles

Cove West

Pile Tip Embedment at 10, 15 and 20 feet
Into the Bedrock

Graphical Summary of Pile Deflection, Shear and
Bending Moment Distribution along Pile Length

SECTION A-A' (Boring B-1)



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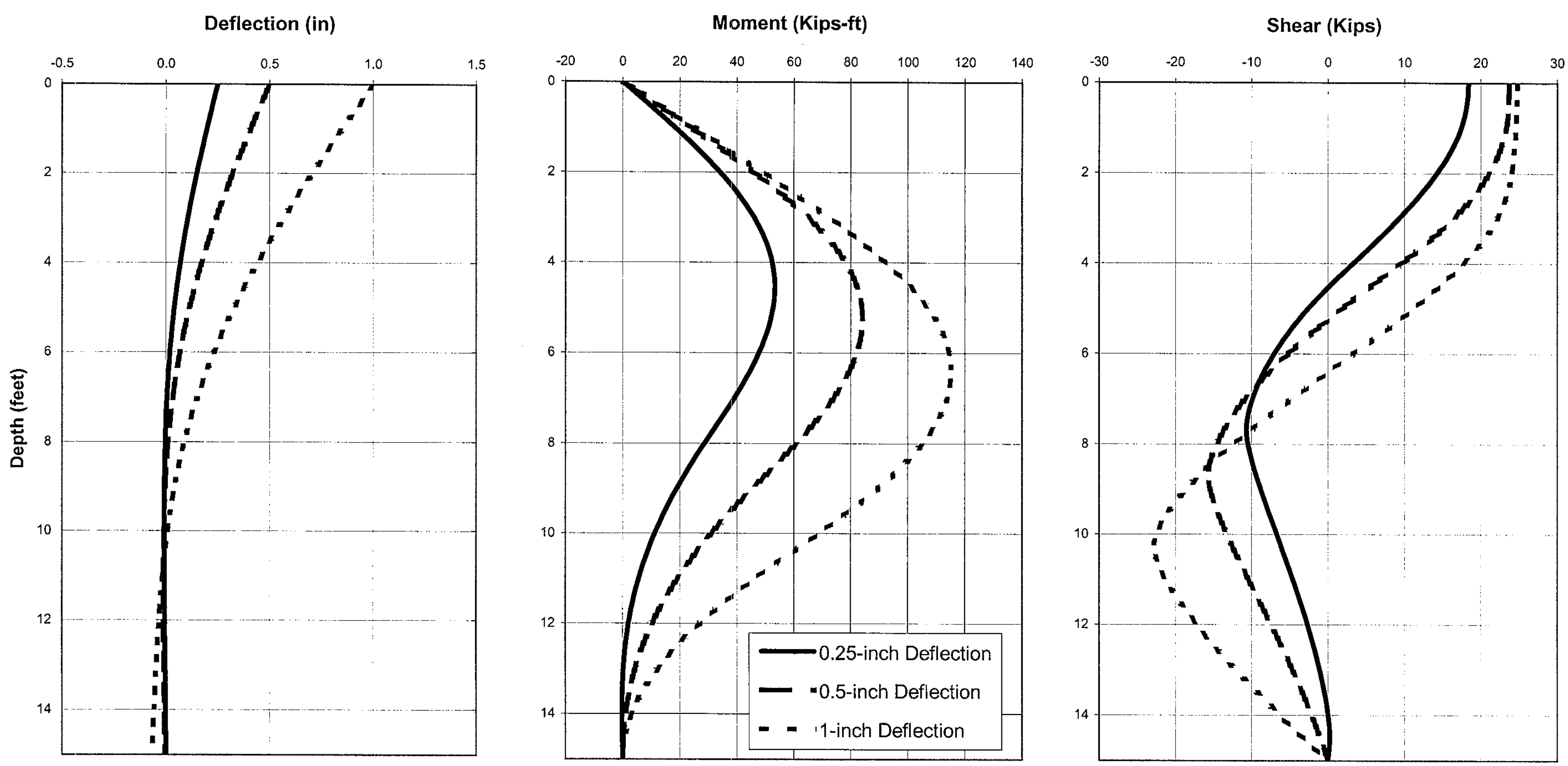
LATERAL LOAD CAPACITIES: 16-INCH CIRCULAR CONCRETE PILE
10-FOOT EMBEDMENT INTO BEDROCK
PRELIMINARY SEAWALL EVALUATION
DANA POINT HARBOR

Project Name:	Dana Point Harbor
Project No.:	600024-003
Date:	4/23/07

Figure 1a

SECTION A-A' (Boring B-1)



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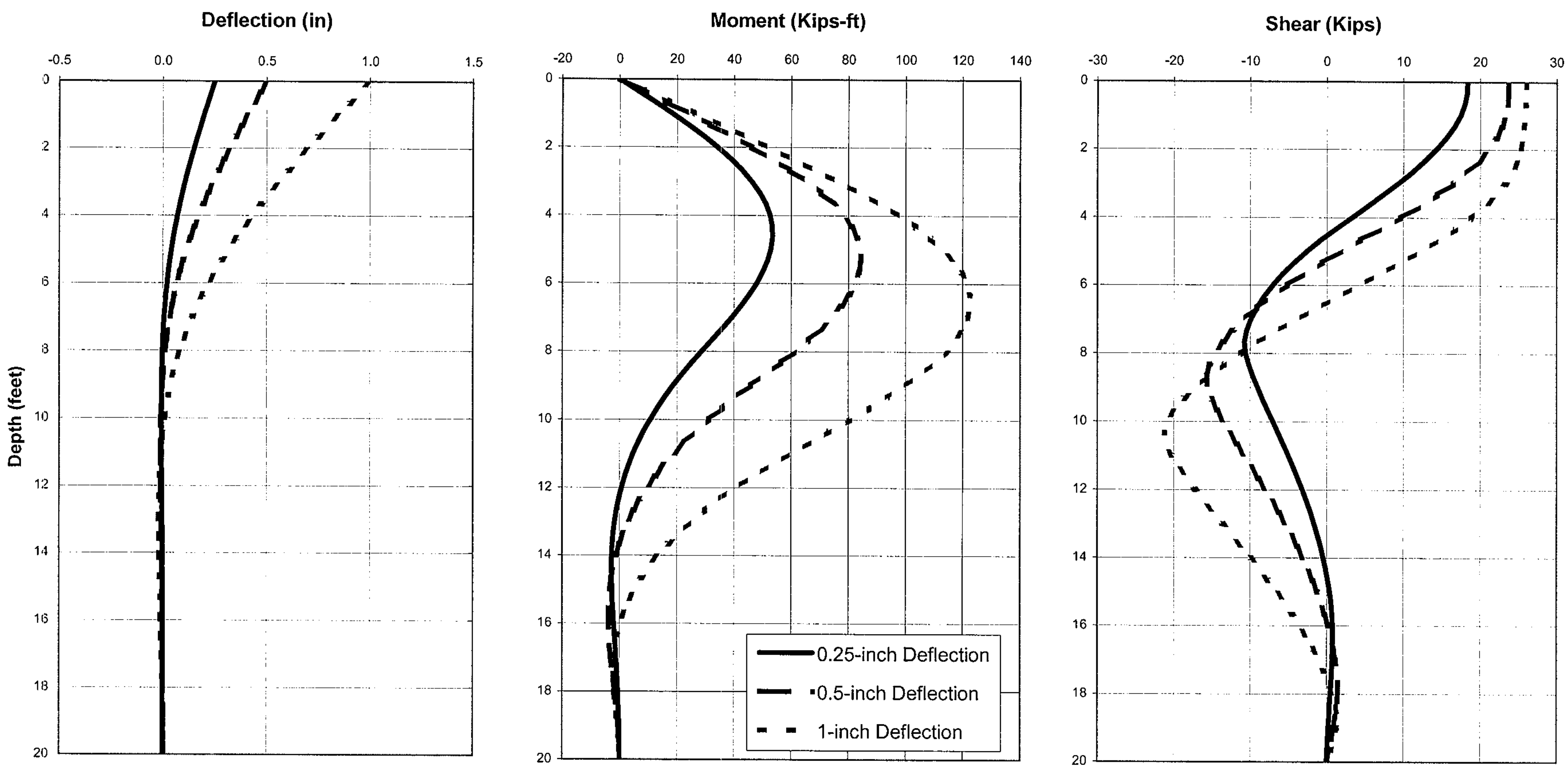
15-FOOT EMBEDMENT INTO BEDROCK
PRELIMINARY SEAWALL EVALUATION
DANA POINT HARBOR

Project Name:	Dana Point Harbor
Project No.:	600024-003
Date:	4/23/07

Figure 1b

SECTION A-A' (Boring B-1)



NOTE: Depth is depth of embedment below harbor bottom (k500pci)

LATERAL LOAD CAPACITIES: 16-INCH CIRCULAR CONCRETE PILE **20-FOOT EMBEDMENT INTO BEDROCK** **PRELIMINARY SEAWALL EVALUATION** **DANA POINT HARBOR**	Project Name: Project No.: Date:	Dana Point Harbor 600024-003 4/23/07

Figure 1c

Lateral Load Capacity

16-inch Diameter Precast Concrete Circular Piles

Island West

Pile Tip Embedment at 10, 15 and 20 feet
Into the Bedrock

Graphical Summary of Pile Deflection, Shear and Bending Moment Distribution along Pile Length

SECTION B-B' (Boring B-3)

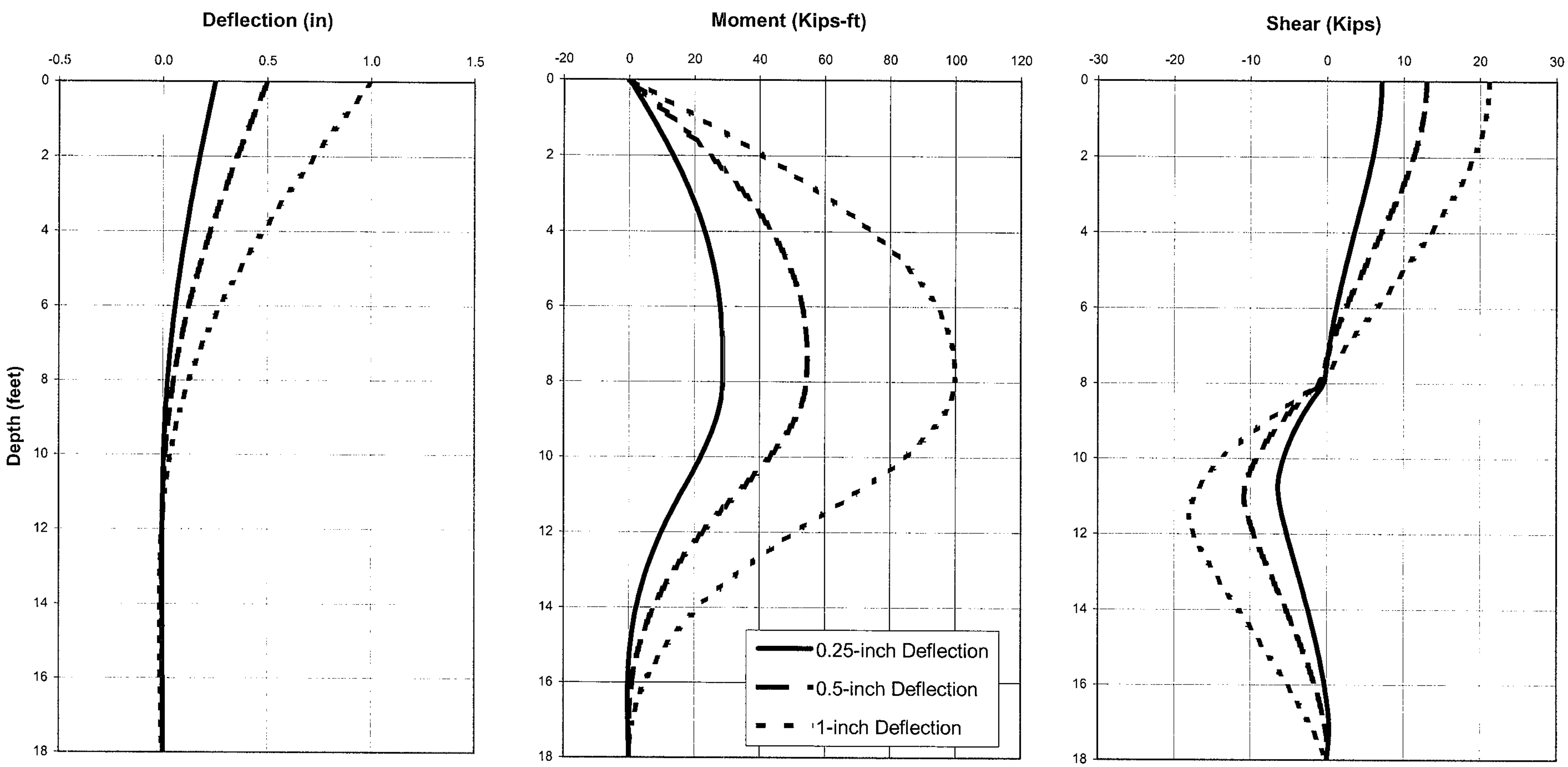


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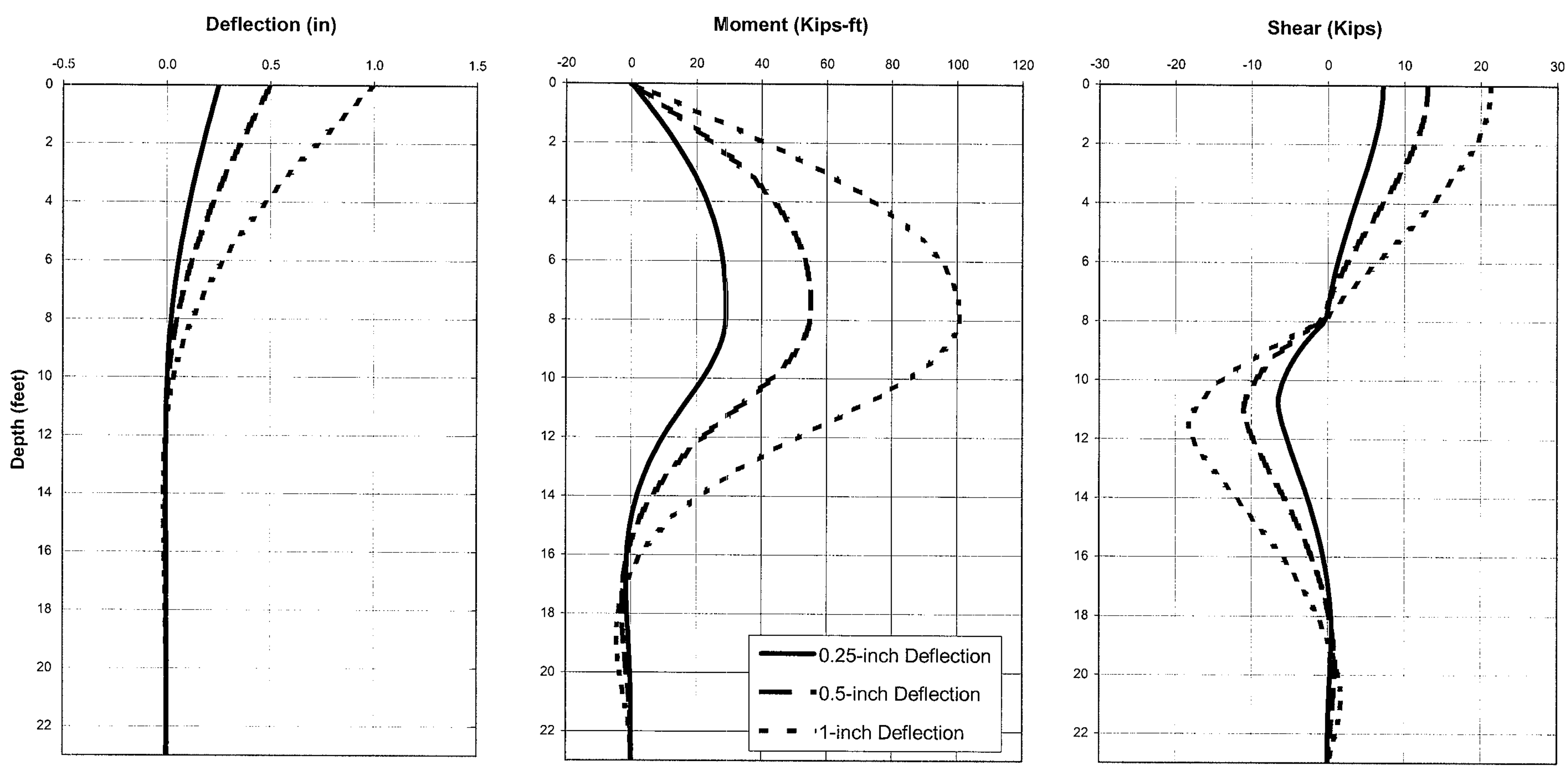
Figure 2b

SECTION B-B' (Boring B-3)



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SECTION B-B' (Boring B-3)

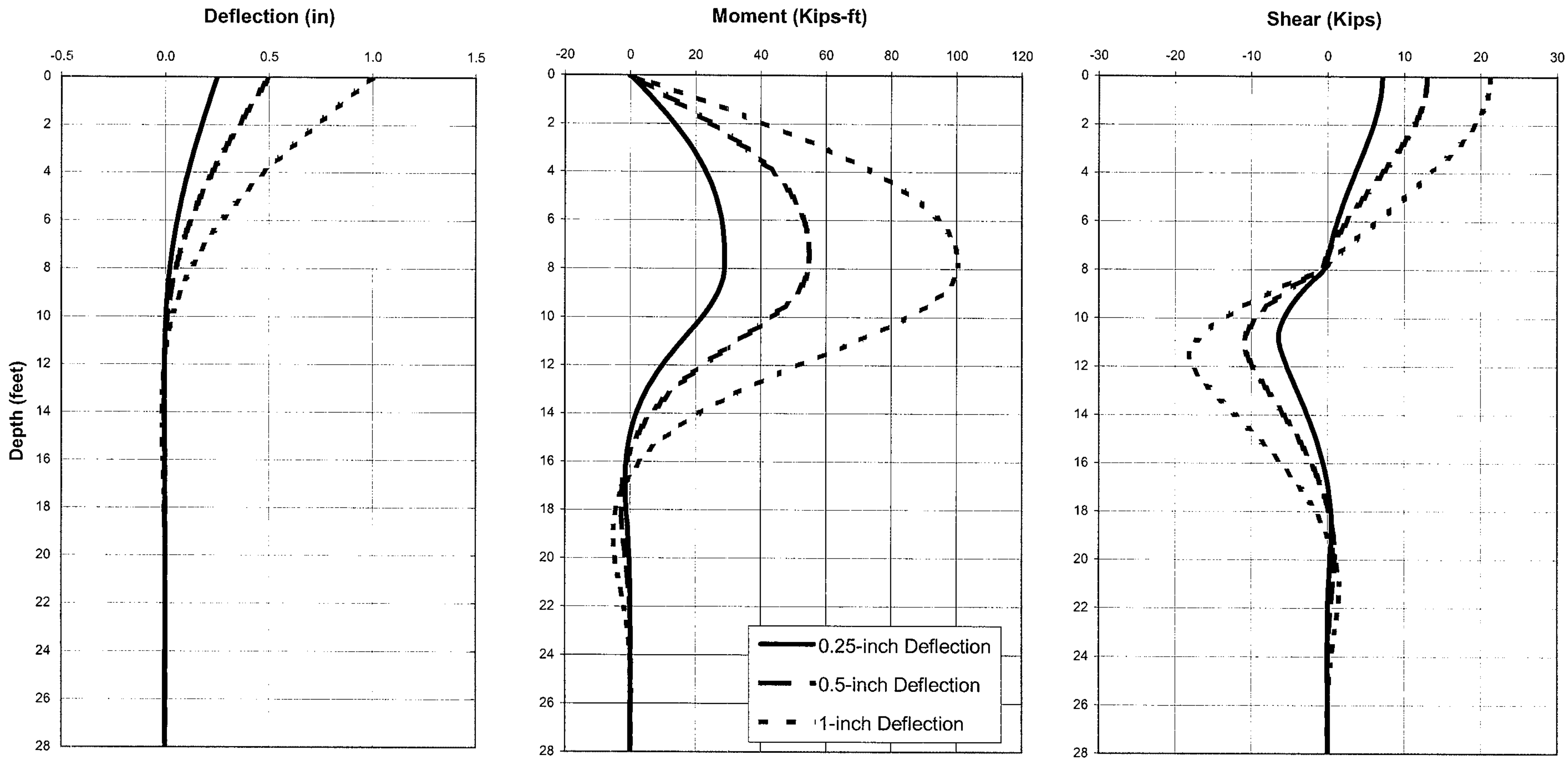

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Figure 3d