GEOTECHNICAL, INC.

23241 Arroyo Vista
Rancho Santa Margarita
CA 92688

voice: 949.888.6513
fax: 949.888.1380
web: www.gmugeo.com

Report of Geotechnical Investigation,
Dana Point Harbor Revitalization Project,
Phase 1, Dana Point Harbor,
County of Orange, California

Prepared for MVE Institutional

July 19, 2013

GMU Project No. 11-161-00

23241 Arroyo Vista
Rancho Santa Margarita
CA 92688

voice: 949.888.6513
fax: 949.888.1380
web: www.gmugeo.com

TRANSMITTAL

MVE INSTITUTIONAL
3 MacArthur Place
Santa Ana, CA 92707

DATE: July 19, 2013

GMU PROJECT: 11-161-00

ATTENTION: Mr. William Koster, Principal

SUBJECT: Geotechnical Investigation, Dana Point Harbor Revitalization Project, Phase 1, Dana Point Harbor, County of Orange, California

WE ARE SENDING THE FOLLOWING:

Electronic copy of our "Report of Geotechnical Investigation, Dana Point Harbor Revitalization Project, Phase 1, Dana Point Harbor, County of Orange, California," dated July 19, 2013.

TABLE OF CONTENTS

TABLE OF CONTENTS (continued)

Description **Page**

TABLE OF CONTENTS (continued)

Description **Page**

TABLE OF CONTENTS (continued)

Description **Page**

INTRODUCTION

PURPOSE

This report presents the results of our geotechnical investigation for the first phase of the Dana Point Harbor Revitalization Project. This first phase is the "Commercial Core" area of the Harbor and includes Planning Areas PA-1 and PA-2. Planning Area PA-1 is the marine services area which currently includes the Embarcadero, the shipyard, and the boat storage area while Planning Area PA-2 includes the Dana Wharf and Mariners Village sections of the Harbor (see Plate 1 – Location Map).

The purpose of this report is to provide a review of the current site plans; provide a summary of our geotechnical investigation, laboratory testing, data analysis, and conclusions; and then provide geotechnical recommendations pertaining to site grading and for the design and construction of the proposed buildings, parking structures, and other site improvements (i.e. roadways, parking lots, site walls, exterior concrete flatwork, etc.). In preparing this report, the following scope of work was performed.

SCOPE

1. Reviewed background information pertaining to the site, including historic aerial photographs, published geologic maps, and previous geotechnical reports for the subject site and surrounding areas by other geotechnical consultants.

2. Performed an initial site reconnaissance to access current surface conditions and mark the site for Underground Service Alert.

3. Conducted a subsurface exploration program that consisted of the advancement of six (6) CPT soundings to depths of 16 to 30 feet and the drilling of nineteen (19) hollow-stem auger borings to depths of 10.5 to 50.5 feet in order to provide both continuous soil data and soil samples for laboratory testing. Borings DH-5, DH-7, DH-10, DH-11, and DH-16 were also used to perform infiltration tests. The borings were logged by our project geologist and samples were collected for laboratory testing.

4. Performed laboratory testing on bulk and undisturbed samples that were collected during our subsurface exploration. Laboratory testing included the determination of in-situ moisture and density, maximum dry density and optimum moisture content, soil gradation, Atterberg Limits, expansion potential, consolidation and shear strength characteristics, chemical and specialty corrosion testing, and R-value.
5. Interpreted and evaluated field conditions and laboratory data.

6. Performed geotechnical engineering analyses using the field and laboratory data in conjunction with the conceptual site plan. The analysis addressed site seismicity, seismic-induced liquefaction and lateral spreading, foundation design including pile analysis, anticipated static settlements, retaining wall evaluation, groundwater concerns, seawall evaluation, and pavement section design.

7. Prepared this report which summarizes the results of our research, subsurface exploration, laboratory and field testing, analyses, conclusions, and recommendations relative to the subject parking structure foundation design and general adjacent site development of the subject project.

SITE LOCATION AND DESCRIPTION

The first phase of the Dana Point Harbor Revitalization Project consists of significant changes to the "Commercial Core" portion of the harbor. The "Commercial Core" is located within the northeast portion of the Dana Point Harbor and is bordered on the north by Dana Point Harbor Drive, on the east by Puerto Place, on the west by Casitas Place, and on the south by a seawall and then Dana Point Harbor. Embarcadero Place and the southern terminus of Street of the Golden Lantern extend from Dana Point Harbor Drive into the subject site. The site is also currently occupied by boat storage parking areas, automotive parking lots, numerous restaurant and retail buildings, and a County of Orange maintenance facility yard. The general location of the site with respect to nearby roadways and landmarks is shown on Plate 1.

The existing boat storage and automotive parking lots are paved with asphalt, have occasional light bollards between the rows of parking stalls, and are surrounded by concrete curbs and gutters. The existing restaurant and retail buildings are one to two stories in height and appear to be of wood-frame construction with conventional foundations.

The majority of the site is relatively flat and level and drains by sheet flow towards the south to existing storm drain catch basins; however, there is an approximately 5-foot -high slope between the boat storage parking lot and Street of the Golden Lantern, a 5- to 10-foot-high slope along the north side of the boat storage parking lot adjacent to Dana Point Harbor Drive, and 1- to 10- foot-high slopes along the east and west sides of Embarcadero Place. In addition, there are minor slopes less than 5 feet in height within the southwest portion of the site between the existing retail buildings and automotive parking lots. Elevations within the site range from a high of approximately 25 feet above mean sea level within the northern portion of the site to a low of approximately 6 feet above mean sea level within the southern portion of the site. The majority of the site is covered by either asphalt pavement or concrete flatwork; however, there are planter and landscape areas that contain flowers, groundcover, shrubs and occasional trees.

BACKGROUND HISTORY AND PREVIOUS GEOTECHNICAL REPORTS

In order to identify and describe the site history and geologic conditions; we reviewed published geologic maps and reports, previous geotechnical reports by other geotechnical consultants for the subject site and entire harbor area, and a previous report for the existing seawalls.

Based on our research, Dana Point Harbor is located within a cove (Dana Cove) that is bordered on the north by cliffs or bluffs that are approximately 100 to 200 feet high and on the west by a hard, resistant promontory of land known as the Dana Point headland. Prior to the development of the harbor, the cove was bordered by a rocky shoreline along the base of the cliffs; however, due to the protection provided by the headland, a sandy shore was able to develop toward San Juan Creek.

Dana Point Harbor was constructed in the late 1960s and early 1970s by the County of Orange and the United States Army Corps of Engineers. It is our understanding that the harbor was constructed by excavating the native soils after the cove was dewatered through the construction of a coffer dam. The construction of the coffer dam included the installation of sheet piling and the placement of fill in a wet condition. After the cofferdam was constructed, the harbor was dewatered and the water basins were excavated to maximum depths of approximately 10 to 12 feet below sea level with the exception of local areas within the northern portion of the harbor where hard bedrock materials were encountered. Artificial fill was then placed in a relatively dry condition up to existing grades and the seawalls, boat ramps and docks, and buildings were then constructed. In addition, a rubble breakwater was constructed along the south side of the harbor to protect it from wave action.

In order to create access to the harbor, the shoreline cliffs were cut back to construct Dana Point Harbor Drive and Street of the Golden Lantern. These slopes were cut to gradients ranging from 1:1 (horizontal to vertical) to 2:1, dependent on their geologic structure and material type.

In 2002, Leighton and Associates performed a preliminary investigation of the subject site for the Dana Point Harbor Master Plan. Leighton found that the most of the Phase 1 area is underlain by approximately 15 to 20 feet of fill with the depths of fill increasing towards the south. However, within the northwest corner of the site, only 1.5 to 3 feet of fill was encountered. The fill materials were reported to consist of sandy clays, clayey sands and silty clays that were found to be medium dense to dense or firm to stiff. A layer of dense, gravelly sand alluvium was also found in Leighton boring B-3 at depths of 15 to 20 feet. Below the artificial fill and local alluvial materials, bedrock materials of the Capistrano Formation were encountered. Leighton reported these materials to consist of very dense sandstones with occasional layers of very hard claystone. The exploration logs and laboratory testing data from this report are included in Appendices A and B, respectively, and the locations of these previous borings are presented on Plate 2.

An evaluation of the existing seawalls was previously performed by Bluewater Design Group in December of 2003. Their evaluation indicated that most of the existing seawalls are “Quay” walls which consist of slightly battered, cantilever, reinforced-concrete gravity walls constructed directly above 1.5H:1V slopes. The slopes are either covered by concrete panels or are constructed with rock rubble. As a result, the wall footings are supported on either fill materials or rock rubble. The walls are not embedded into the ground and thus rely on their own weight, the weight of the soil over the heel, and the friction between the bottom of the footings and the underlying soil or rip-rap to resist overturning and sliding forces. Most of the Quay walls have a height of 5 feet; however, some local sections have a height of 9 feet.

The report by Bluewater Design group also indicated that the north and south sides of the public boat launch ramp are supported by conventional cantilever retaining walls that range from 2 to 15 feet in height with footings founded into fill materials.

In November of 2005, Diaz-Yourman & Associates performed a geotechnical investigation for the rehabilitation of the Dana Point Harbor boat launch ramp. Diaz-Yourman found that boat ramp area is underlain by approximately 15 to 20 feet of fill. The fill materials were reported to consist primarily of silty sands and silty gravels with occasional layers of clayey sand and sandy silt that were found to be medium dense to dense. Below the artificial fill, bedrock materials of the Capistrano Formation were encountered. Diaz-Yourman reported these materials to consist of very dense, fine to coarse grained sandstones. The exploration logs and laboratory testing data from this report are included in Appendices A and B, respectively, and the locations of these previous borings are presented on Plate 2.

In 2008, Leighton and Associates performed a geotechnical engineering exploration and analysis for all of the existing seawalls within the harbor and to provide recommendations for the design and construction of pedestrian platform structures and guide piles within the marina for new boat docks. Leighton expanded their subsurface exploration to include the southern and northwestern portions of the harbor. They found that the northern "Cove" portion of the harbor is underlain by 10 to 20 feet of fill while the southern "Island" portion of the harbor is underlain by 23 to 30 feet of fill. The fill materials were reported to consist of medium dense, fine to medium grained sands with varying clay contents. Below the artificial fill materials, sandstone and claystone bedrock materials of the Capistrano Formation were encountered as described previously. The exploration logs and laboratory testing data from this report are included in Appendices A and B, respectively.

Diaz-Yourman recently (October, 2012) drilled a boring within the existing shipyard to provide geotechnical recommendations for a new crane to be constructed near the existing quay seawall. Diaz-Yourman has not yet completed their geotechnical report; however, they provided us with a draft copy of the log of this boring. The boring log is included in Appendix A and the location of this recent boring is presented on Plate 2.

AERIAL PHOTOGRAPHY REVIEW

An aerial photo review was performed for the subject site in order to assess historical land use and site development. Continental Aerial Photo provided 20 sets of stereo-paired air photos spanning from 1952 through 1999. Photos taken prior to development of the harbor area show an undeveloped cliff bordered by a rocky shoreline and a relatively natural cove. In 1967, two jetties were constructed on the east and west sides of the cove. By 1970, the alteration of the cove into a man-made harbor was nearing completion and the roadways had been graded. The photos indicate that Dana Point Harbor Drive and the northerly areas of the harbor (generally parking lot and boat storage) are likely underlain by bedrock from the cut operation of the shoreline cliff. By 1975, the harbor appears to be in essentially the same as it is currently, with all existing buildings constructed and paved areas completed. Photos reviewed after 1975 show no significant changes to the area.

PLANNED IMPROVEMENTS AND GRADING

Based on our review of the preliminary site plans, it is proposed to revitalize the primary commercial portion of the Dana Point Harbor which includes the marine services area (PA-1) and the Dana Wharf and Mariners Village sections (PA-2) of the Harbor.

The revitalization of the marine services area (PA-1) will include the removal of existing buildings and structures and a portion of the existing parking lot to construct a new 50,000 square feet Dry Boat Storage Building (Building #M1) with an attached Office/Retail building and maintenance canopy. The boat storage building will be built partially out over the existing bay on piles and will contain an automated crane to transport boats from the bay to their storage locations. The Office/ Retail building will be located along the northwest side of the Dry Boat Storage Building and will include a lounge/waiting area, restrooms, and office and retail space. The maintenance canopy will be located along the southeast side of the Dry Boat Storage Building and will simply consist of a concrete slab covered by a shade canopy. It is expected that the Dry Boat Storage Building will be of steel-frame construction with the lowermost floor slabs constructed both on-grade and out over the water of the Dana Point Harbor.

Other changes to the marine services area will be the removal of the current access road (Embarcadero Place) and the construction of a new entrance off of Puerto Place. To construct this new entrance, a portion of Puerto Place near Dana Point Harbor Drive will need to be widened. The layout of the parking lots will also be revised and it is likely that the existing pavement sections within the parking lots and drive isles will need to be removed and replaced with new sections due to their present poor condition. The existing public boat launch ramp and shipyard will be protected in-place.

The revitalization of Dana Wharf will consist primarily of minor exterior improvements to the existing buildings and surrounding landscaping; however, the southern portion of Building 5 will be demolished and removed while Building 4 will receive an addition along its north side. All of the buildings (1 through 5) will then be upgraded with new siding and stone accents and new roofs and the restaurant buildings will be provided with new grease interceptors and associated plumbing. It is expected that the addition to Building 4 will be of wood- or steel-frame construction with the floor slab constructed on-grade. It is also expected that this addition will be supported on conventional shallow foundations that are similar to the existing foundations. The existing delivery areas and trash enclosure areas within the Wharf Area will be removed and replaced and new exterior lighting will be provided. The existing concrete walkways and patios may also be removed and replaced with new concrete walkways and patios or just repaired as needed.

The existing buildings within the Mariners Village area will be demolished and removed to allow construction of a new three-story Retail/Office building (Building 6), three new two-story Retail/Restaurant buildings (Buildings 7, 8, and 9), two new one-story Restaurant buildings (Buildings 10 and 11), a new two-story Locker/Public Restroom building (Building 12), and a two level parking structure (Parking Deck P1). In addition, a parking podium (Podium P2) will be constructed between the parking structure and Buildings 6, 7, and 8 and will create drive aisles, drop-off-areas, and parking spaces along the north sides of the buildings. The upper level of the podium will wrap between and around the buildings to create elevated outdoor seating decks along the south sides of the buildings. A ramp supported by retaining walls will lead up to the podium from Street of the Golden Lantern. The second levels of Buildings 6 and 7 will also be connected by a pedestrian bridge that will be constructed over the Dana Wharf access driveway. It is expected that these buildings will be of wood- or steel-frame construction with the lowermost floor slabs constructed on-grade while the parking structure and podium will be constructed with reinforced concrete slabs and columns. In addition, the proposed Locker/Public Restroom Building (Building 12) will be constructed partially with retaining walls due to local variations in ground surface elevations.

Other changes to the Dana Wharf and Mariners Village areas will consist of almost the complete removal of existing Street of the Golden Lantern and its replacement with the ramped driveway and podium structure. New concrete walkways, stairways, patios and site walls will be constructed around the new buildings. In addition, Casitas Place and the walkway along the south side of Dana Point Harbor Drive will be widened and the layout of the existing parking lots will be revised. Due to their poor existing condition, it is likely that the existing parking lot and drive isle pavement sections will need to be removed and replaced with new sections.

Through the majority of the site, proposed grades will remain essentially the same as existing grades with only minor cuts and fills of a few inches up to 1 to 2 feet being required. However, local areas will require more significant grading. The proposed parking structure will require cuts of approximately 2 to 4 feet to reach proposed grades. Buildings 10 and 12 will require fills of up to a foot and cuts of up to 5 feet to reach proposed grades and Building 11 will require fills of only a few inches up to approximately 3 feet. The parking lots and drive isles around these buildings will also require similar cuts and fills. Cuts of up to approximately 8 feet will be required to remove Embarcadero Place while fills of up to approximately 5 feet will be required to widen the south side of Dana Point Harbor Drive and the new entrance off of Puerto Place. Small slopes ranging up to approximately 5 feet in height will be constructed to the east and south of proposed Building 12 and to the east and west of Building 10. These slopes will be constructed at a slope ratio of 2:1, horizontal to vertical, or less.

SUBSURFACE EXPLORATION

Our recent subsurface investigation consisted of the drilling of nineteen (19) hollow-stem auger drill holes (DH-1 through DH-19) to depths of 10 to 50.5 feet, the drilling of one (1) hand-augered drill hole (DH-20) to a depth of 4.5 feet, and the excavation of four (4) shallow pavement core holes (C-1 through C-4) to depths of 1.5 to 3 feet to obtain bulk and drive samples for geotechnical testing, to observe depths to groundwater, and to observe the thicknesses of the existing pavement sections. We also advanced six (6) CPT soundings (CPT-1 through CPT-6) to depths of 16 to 30 feet to obtain continuous geotechnical information on the subsurface soil and bedrock materials. Due to the very dense/hard condition of the bedrock materials underling the site, we were only able to advance one of the CPT soundings (CPT-2) more than 1 to 2 feet into the bedrock. Secondary shallow drill holes were drilled adjacent to drill holes DH-5, DH-7, DH-10, DH-11, and DH-16 in order to perform infiltration tests.

All of the drill holes were logged by a Certified Engineering Geologist and bulk and undisturbed samples of the excavated soil and bedrock materials were collected for laboratory testing. “Undisturbed” drive samples were taken using a 3.0-inch outside diameter split spoon sampler which contained 2.416-inch-diameter brass sample sleeves 6 inches in length. Standard Penetration Tests (SPT) using a 2.0-inch outside diameter split spoon sampler without liners were also taken in drill holes below a depth of 10 feet in DH-1 through DH-19, at selected depths in between the relatively undisturbed samples. Blow counts recorded during sampling from the drive samplers are shown on the drill hole logs including uncorrected SPT blow counts (i.e., “N” values). The logs of each boring are contained in Appendix A-1 and the Legend to Logs is presented as Plate A-1. CPT soundings were performed with a 30-ton CPT rig and a 15-cm^2 cone with readings taken every 2 cm. The CPT logs and data are contained in Appendix A-2. The previous borings and CPT soundings performed by Leighton and Diaz-Yourman are contained in Appendix A-3.

The approximate locations of the drill holes, pavement core holes, and CPTs are shown on Plate 2 – Geotechnical Map. The locations of the previous borings and CPT soundings by Leighton and Diaz-Yourman are also shown on Plate 2.

INFILTRATION TESTING

Infiltration testing was performed in general accordance with the Santa Ana Regional Water Quality Control Board Technical Guidance Document (TGD) Appendices dated March 2011, utilizing the shallow percolation test procedure contained in Section VII.3.8. To comply with the requirements of the TGD, five (5) 10-inch-diameter test holes were excavated adjacent to drill holes DH-5, DH-7, DH-10, DH-11, and DH-16 to depths of approximately 3 to 4 feet using a hollow stem auger drill rig. The infiltration test hole locations are shown for ease of reference on the attached Geotechnical Map, Plate 2.

Logs for DH-5, DH-7, DH-10, DH-11, and DH-16 are contained within Appendix A-1 and indicate that the site is underlain by approximately 15 to 20 feet of engineered fill overlying bedrock materials of the Capistrano Formation. The fill materials are highly variable and consist of intermixed layers of silts, clays, silty sands and clayey sands while the bedrock materials consist of hard to very hard and massive sandstones with occasional thick layers of moderately hard to hard claystones and siltstones. The holes were drilled to depths of 3 to 4 feet and infiltration was monitored from depths ranging from approximately 2 to 4 feet below grade which correspond to the infiltration zone of a potential infiltration system.

LABORATORY TESTING

Laboratory testing for the subject investigation was performed to determine soil engineering classifications and properties. Recent and previous testing included the following: in-place moisture and dry density, maximum dry density and optimum moisture content, particle size distribution, Atterberg limits, chemical corrosion suite, consolidation characteristics, undisturbed and remolded shear strengths, subgrade R-Values, and expansion index tests. Laboratory procedures and recent test results are presented in our Appendix B-1 – GMU Geotechnical Laboratory Procedures and Test Results. Previous laboratory test results from Leighton and Diaz-Yourman are presented in Appendices B-2 and B-3. Pertinent laboratory test data is also shown on our recent drill hole logs and previous boring logs.

Laboratory test results on samples collected at the site indicate that very low to medium expansive soils are present. Visual descriptions indicate that the on-site engineered fill materials consist of clayey sands, sandy clays, and sandy silts while the underlying bedrock materials consist primarily of hard to very hard sandstones with occasional thick and moderately hard to hard claystone and siltstone layers. Given the exploration and laboratory data, it is our opinion that the proposed improvements should be designed assuming a medium expansion potential.

The results of chemical testing indicate that the on-site soils will be severely corrosive to ferrous metals. The results of sulfate tests indicate that the site will have a negligible to moderate sulfate exposure to concrete as defined by the CBC.

GEOLOGIC FINDINGS

REGIONAL GEOLOGIC SETTING

The project area is contained within the northwestern portion of California's Peninsular Ranges

Province and at the southeastern extremity of the Los Angeles basin. The Peninsular Ranges are characterized by northwest trending, parallel, fault-bounded mountain ranges separated by valleys. This sequence of mountain ranges and valleys extends from the northern side of the Los Angeles Basin southward into Baja California. Dana Point Harbor is located along the shoreline below elevated marine and non-marine terraces that flank the southeastern corner of the San Joaquin Hills, which are a northwest trending elevated area that extends from Newport Beach southward to Dana Point.

LOCAL GEOLOGY AND SUBSURFACE SOIL CONDITIONS

Published geologic maps indicate that prior to development the site consisted of a natural cove that was protected by a hard, resistant promontory of land to the west known as the Headlands. The cove was bordered by a rocky shoreline along the base of steep sea cliffs. The sea cliffs are comprised of marine sedimentary rocks of the Capistrano Formation that are capped by marine and non-marine terrace deposits. The base of the sea cliffs were mantled by talus deposits and local deposits of artificial fill while the bottom of the cove was covered by beach deposits.

As described previously in this report, the harbor was constructed by dewatering the cove, excavating the native soils along the base of the cliffs and within the cove, replacing the excavated materials as compacted fill, and creating cut slopes to create roadways to the harbor. Based on the results of our recent subsurface exploration, the subject site is underlain by approximately 15 to 20 feet of artificial fill and then by bedrock materials of the Capistrano Formation with the exception of the northwest and northeast corners of the site where bedrock materials were encountered within approximately 1 to 2.5 feet of the existing ground surface. In general, the depths of fill across the site increase in a southerly direction toward the bay.

It should be noted that Leighton identified a 5-foot-thick layer of dense to very dense gravelly sand alluvium between the fill and bedrock materials within their boring B-3. This may be either an isolated layer of competent native alluvial material that was left in-place during original grading, or it may actually be a layer of artificial fill or weathered sandstone bedrock that was misidentified as alluvium.

Detailed descriptions of the geologic materials beneath the site as observed during our recent subsurface exploration are described below.

Artificial Fill (Qaf)

The artificial fill materials within the site originated from the both the native beach deposits and bedrock within the cove and from the talus deposits and bedrock materials along the base of the sea cliffs. As a result of the fill materials being derived from a variety of different geologic units, the fill materials are highly variable and consist of frequently alternating layers of clayey sands, silty sands, sands, sandy clays, and sandy silts. In general, the granular sand materials were found to be medium dense to dense while the fine grained clay and silt materials were found to be predominantly firm to very firm. In addition, our laboratory testing indicates that the fill materials have varying amounts of compressibility and hydro-collapse.

Capistrano Formation (Tc)

Capistrano Formation bedrock was encountered below the fill in all of our deeper drill holes and in all of our CPT soundings. The bedrock was observed to consist predominantly of hard to very hard, fine to coarse grained, massive sandstones with occasional thickly bedded layers of moderately hard to hard, gray to very dark gray claystones and siltstones.

GEOLOGIC STRUCTURE

We were unable to accurately measure the strike of any bedding planes within our small diameter drill holes; however, bedding observed within samples recovered indicate bedding dips of horizontal to gently dipping (up to 15 degrees). Regional geologic maps (Edgington, 1974) indicate that the bedrock materials in the vicinity of the site dip towards the northeast which is considered favorable with respect to the stability of the cut slopes located across Dana Harbor Drive. Visual observation of these cut slopes during our field exploration program indicate these slopes do not show signs of gross or significant surficial failure, and appear to expose massive sandstone and conglomeratic sandstone with very rare faint bedding that appears to be horizontal or northerly dipping. In addition, these cut slopes lie on the other side of Dana Point Harbor Drive and are at least 100 feet from the subject site; therefore, any instability of these slopes is not expected to have an impact on the proposed development.

GROUNDWATER

Groundwater was encountered within our recent drill holes and CPT soundings at elevations that primarily ranged from 8 feet below mean sea level (MSL) to 6 feet above MSL (depths of 8 to 14 feet below existing grades). However, within one of our CPT soundings (CPT-6), groundwater was encountered at a depth of only 5 feet below existing grade. This is likely due to the proximity of the CPT sounding to the adjacent boat launch ramp and due to high tide conditions at the time the test was taken.

Groundwater elevations across the site are controlled by the elevation of the water within the adjacent bay but are also somewhat influenced by the pre-development topography, with lower elevations found closest to the seawalls. It should be noted that the groundwater elevations measured during our exploration were affected by the time of day as it relates to the local tidal cycle, and therefore should be assumed to fluctuate with the tides, the lunar cycle, and recent rainfall events.

In order to better evaluate the groundwater data collected during our investigation, these depths to groundwater were compared to the depth of historically high groundwater shown on within the Seismic Hazard Zone Report for the Dana Point Quadrangle (CDMG, 2001). These maps indicate a historical high groundwater of 5 feet b.g.s. which corresponds with the highest elevation of groundwater found during our investigation.

Based on the above findings, groundwater may be encountered as high as 5 feet b.g.s – although this occurrence is anticipated to be localized and during high tide conditions. Consequently, the groundwater may impact proposed corrective grading (i.e. at the bottom of the removals) as well as utility trenches deeper than 5 feet b.g.s.

FAULTING AND SEISMICITY

The site is not located within a published Alquist-Priolo Earthquake Fault Zone, and no known active faults are shown on current geologic maps for the site. Plate 4 shows the site location with respect to regional seismic sources. The nearest known active fault is the offshore segment of the Newport-Inglewood fault, which is located approximately 3.9 kilometers southwest of the site and is capable of generating a maximum earthquake magnitude (M_w) of 7.1. The site is also located within 11.3 kilometers of the surface projection of the San Joaquin Hills Blind Thrust, which is capable of generating a maximum earthquake magnitude (M_w) of 6.6. Given the proximity of the site to these and numerous other active and potentially active faults, the site will likely be subject to earthquake ground motions in the future.

The site structures are underlain by hard to very hard bedrock of the Capistrano Formation and a relatively shallow mantle of engineered fill. Consequently, the stiffness and shear wave velocity of the Capistrano Formation bedrock will control the seismic response. We were able to determine the shear wave velocities of the fill materials by performing three seismic CPT soundings (CPT-1, CPT-3 and CPT-5). However, since only one of the CPT soundings was able to penetrate into the bedrock, the shear wave velocities of the bedrock layers could not be determined by the seismic CPT soundings. Therefore, the shear wave velocities of the bedrock were estimated using the SPT values obtained in our deep exploratory drill holes. The shear wave velocities of the fill and bedrock were then used to determine that Site Class C (very dense soil or soft rock) is the most applicable to the site (i.e., shear wave velocity for upper 100 feet is estimated to be greater than 1200 ft/sec).

In order to evaluate the likelihood of future earthquake ground motions occurring at the site, a probabilistic seismic hazard analysis (PSHA) of horizontal ground shaking was performed using the commercial computer program EZ-FRISK ver. 7.43. The PSHA utilized seismic sources and attenuation equations consistent with the 2008 USGS National Seismic Hazard Mapping Project. Assuming a risk level of 10 percent probability of exceedance in 50 years (i.e., ~ 475 year ARP), the PHGA is 0.32g.

For the purposes of our liquefaction analysis, the PSHA discussed above was also deaggregated to determine the mode magnitude and mode distance. The deaggregation resulted in a mode magnitude of 6.8 and mode distance of 3.8 km.

If requested by the project structural engineer, GMU can also provide a site-specific ground motion hazard analysis per ASCE 7-05 Sections 21.2, 21.3, and 21.4.

SEISMIC HAZARD ZONES

The subject property is not located within an area mapped as having the potential for seismic-induced landsliding; however, it is located within an area mapped as having the potential for seismic-induced liquefaction as shown on the reference (2) Seismic Hazard Zone Map for the Dana Point Quadrangle.

GEOTECHNICAL ENGINEERING FINDINGS

LIQUEFACTION, SEISMIC SETTLEMENT, AND LATERAL SPREADING

Liquefaction Investigation

The site is located within a zone mapped as having the potential for earthquake induced liquefaction. In addition, groundwater was observed at depths of approximately 8 to 14 feet and granular soils were encountered below the groundwater. Therefore, liquefaction and related hazards were quantitatively evaluated utilizing the subsurface data from our CPT soundings and the previous CPT soundings by Leighton.

Design Earthquake and Mode Magnitude

Based on our site specific PSHA with deaggregation, a PGA of 0.32g, Modal Magnitude of 6.8, and modal distance of 3.8 km were calculated for this study. However, since the Seismic Hazard Zone Report for the Dana Point Quadrangle (CDMG, 2001) indicates that the subject site lies within an area that is expected to have a PGA of 0.35 for soft rock conditions, we conservatively used a PGA of 0.35g in our liquefaction analysis.

Design Groundwater Level

The referenced seismic hazard evaluation report indicates a historically high groundwater level of 5 feet b.g.s. Actual groundwater levels encountered during our recent exploration ranged from approximately 8 to 14 feet below existing site grades with a local occurrence of 5 feet b.g.s. Therefore our analysis was performed using the worst case condition (5 feet b.g.s.).

Liquefaction Analyses

GMU utilized Cliq to evaluate CPT data for liquefaction. Cliq is a commercial computer software program that applies the latest NCEER methods for liquefaction analysis including post-earthquake settlement and lateral displacement.

Liquefaction, Seismic Settlement, and Lateral Spreading Potential

Our analysis indicates that relatively thin, discrete zones within the zone of artificial fill below the water table may be subject to liquefaction during a design seismic event. Based on our analysis, the site has a low potential for any adverse effects of liquefaction due to seismic-induced settlement. Our liquefaction seismic settlement calculations indicate approximately 0 to 0.6 inches of settlement could occur during a design earthquake.

However, the site has a moderate to high potential for adverse effects due to seismic-induced lateral spreading. Our calculations indicate that lateral displacements within 20 to 40 feet of the seawalls could range from approximately 20 to 30 inches while laterals displacements within areas located more than 40 feet from the seawalls could range from non-existent to 6 inches or more. The results of our analyses are presented in Appendix C.

STATIC SETTLEMENT/COMPRESSIBILITY

As described previously, the fill materials are highly variable and consist of frequently alternating layers of clayey sands, silty sands, sands, sandy clays, and sandy silts. In general, the granular sand materials were found to be medium dense to dense while the fine grained clay and silt materials were found to be predominantly firm to very firm. In addition, our laboratory testing indicates that the fill materials have varying amounts of compressibility and hydro-collapse. Without mitigation, total static settlements can be expected to range from less than ½ of an inch to an inch below the proposed buildings and up to 1.5 inches below the proposed entrance ramp.

SOIL EXPANSION

The expansion potential of the on-site fill and bedrock materials were assessed based on visual classifications, particle size distributions, Atterberg limits, expansion index, previous studies, and our local experience. The laboratory test summary table is contained in Appendix B-1. The artificial fills mantling the site are highly variable with expansion potentials that range from very low to medium. The sandstone bedrock materials are expected to exhibit a very low expansion potential while the claystones and siltstone are expected to possess a medium to high expansion potential. Since the near surface fill materials have a predominant medium expansion potential, the design of building slabs and exterior hardscape features that will be in contact with these materials should be designed assuming a medium expansion index.

SOIL CORROSION

To evaluate the corrosion potential of the on-site soils to both ferrous metals and concrete, representative samples were tested for pH, minimum resistivity, soluble chlorides, and soluble sulfates and combined with existing results from previous investigations. The results of chemical testing contained in Appendix B indicate that the on-site soils should be considered corrosive to severely corrosive to ferrous metals and possess a negligible to moderate sulfate exposure to concrete. In addition, the proposed building and structures will be exposed to seawater. Therefore, a moderate exposure to sulfates should be anticipated for concrete placed in contact with on-site soils.

SOIL INFILTRATION RESULTS

As described previously, infiltration testing was performed within the site in general accordance with the Santa Ana Regional Water Quality Control Board Technical Guidance Document (TGD) Appendices dated March 2011, utilizing the shallow percolation test procedure contained in Section VII.3.8. The results of the infiltration testing indicate infiltration rates ranging from 0.57 to 1.18 inches per hour with an average rate of 0.83 inches per hour.

EXCAVATION CHARACTERISTICS

Rippability

The artificial fill materials underlying the site can be easily excavated with conventional grading equipment such as dozers, loaders, excavators, and backhoes. Shallow bedrock materials within the northeast and northwest corners of the site, if encountered, may require ripping with dozers.

Trenching

We expect that excavation of new utility trenches can be accomplished utilizing conventional trenching machines and backhoes. Trench support requirements will be limited to those required by safety laws or other locations where trench slopes will need to be flattened or supported by shoring designed to suit the specific conditions exposed.

Volume Change

In order to aid planning for the anticipated grading, we estimate that the change in volume of on-site disturbed surficial fills that are excavated and placed as new compacted fill at an average relative compaction of 92% will result in volume losses that will range from approximately 5 to 10 percent. For rough planning purposes only, an average volume loss of 7.5 percent may be assumed.

CONCLUSIONS

DEVELOPMENT FEASIBILITY

Based on the geologic and geotechnical findings, it is our opinion that proposed construction is feasible from a geotechnical standpoint. However, there are several hazards that must be mitigated to provide long-term site stability and proper support of proposed structures. The subject property will be suitable for the proposed grading and construction provided that the site hazards are mitigated in accordance with the recommendations of this report and with the City of Dana Point and County of Orange grading and building requirements. It is also the opinion of GMU Geotechnical that proposed grading and construction will not adversely affect the geologic stability of adjoining properties provided grading and construction are performed in accordance with the recommendations provided in this report.

MITIGATION OF SITE HAZARDS

As described previously, the site is underlain by a shallow groundwater table and by artificial fill materials that are highly variable and consist of frequently alternating layers of clayey sands, silty sands, sands, sandy clays, and sandy silts. In general, the granular sand materials were found to be medium dense to dense while the fine grained clay and silt materials were found to be predominantly firm to very firm. Based on our analysis, these fill materials are highly variable from an expansion, settlement, and bearing perspective and subject to over several inches of total settlement. In addition, the fill soils contain occasional thin zones of medium dense sands and silts that have the potential for liquefaction-induced lateral spreading. Lateral displacements within 20 to 40 feet of the seawalls could range from approximately 20 to 30 inches while laterals displacements within areas located more than 40 feet from the seawalls could range from non-existent to approximately 6 inches. Based on these anticipated conditions, the following methods of mitigation were considered for the site.

Driven Piles

Consideration was given to using deep foundations such as driven piles beneath the proposed structures that would extend through the artificial fill materials and into the underlying competent bedrock; however, there are several inherent problems relative to their use:

1. They would be difficult to install due to the hard to very hard nature of the bedrock and expensive pre-drilling and the use of hardened pile tips would likely be required. In addition, it is expected that a certain number of piles would become distressed as a result of the amount of force that would be required to drive them to depth. These distressed piles would need to be replaced and/or supplemented with additional piles.

2. The pile driving would result in adverse vibration of nearby ground surfaces and structures. This is a particular concern for the foundations of proposed Building #6 which will be located in close proximity of existing Building #5 that is to be protected in-place.

3. Driven piles will not mitigate the potential for liquefaction and liquefaction-induced lateral

spreading. As a result, the buildings would need to be constructed with structural slabs supported by grade beams. In addition, the potential settlement and lateral movement of the liquefiable zone will exert large down drag loads and lateral forces on the piles. These extra loads will result in deeper, larger, and more frequent piles. Furthermore, appurtenant structures and utility lines within the site would still be affected by settlement and lateral spreading unless these structures are also supported by deepened foundations.

Auger Cast Piles

Auger cast piles could also be used and their installation would not cause significant vibration of the surrounding ground and structures. However, due to the shallow depths to groundwater, their installation would require expensive specialized drilling and concrete placement techniques. In addition, auger cast piles would not mitigate the potential for liquefaction; therefore, the same adverse affects of liquefaction-induced settlement and lateral spreading would exist as described above for driven piles.

Rammed Aggregate Piers

As an alternative to deepened foundations, engineered rammed aggregate (stone) piers may be used within the site to: 1) mitigate the potential for liquefaction-induced lateral spreading and 2) mitigate the problems associated with the fill variability and its support of the building foundations.

This procedure consists of driving a hollow mandrel into the ground at frequent intervals using a powerful static down force augmented by high frequency vertical impact energy. Stone aggregate is then placed inside the hollow mandrel and densely compacting in successive lifts using a specialized beveled tamper. The stone aggregate is typically placed in 1 to 2 foot thick lifts. Each lift is thoroughly compacted before the next lift is placed. This procedure not only creates stone piers but, by using a specialized beveled tamper, also laterally compacts the soil between the stone piers.
The benefit of this method of mitigation is that the proposed buildings may be constructed using conventional spread footings provided that an adequate amount of ground improvement is achieved. The use of stone piers below the buildings would also increase the bearing capacity of the existing artificial fill materials and thus reduce the size and cost of the foundations. Furthermore, verification testing of the subsurface soils beneath the site after ground modification has been performed would provide a relatively high level of confidence in the quality of the soils and stone piers that will underlie the site and proposed buildings.

Recommended Approach

In light of the available mitigation design alternatives discussed above, it is the professional opinion of this firm that the use of engineered rammed aggregate piers will provide the best method of supporting the proposed buildings and mitigating the existing seismic hazards within the site. Recommendations for these aggregate piers are provided in the following section of this report.

GROUND MODIFICATION RECOMMENDATIONS

LIQUEFACTION-INDUCED LATERAL SPREADING MITIGATION

Engineered rammed aggregate piers should be used to create a zone of modified ground that will no longer be prone to liquefaction-induced lateral spreading. As a result, the ground surfaces behind the zone of improvement would no longer be subject to lateral movements and would be able to adequately support the proposed buildings.

Based on our stability analysis, this zone of ground modification should be at least 20 feet wide in order to mitigate the adverse effects of lateral spreading on the new buildings. The ground modification zone should be located 15 feet away from the existing quay seawalls. The results of our stability analysis are provided in Appendix D.

The rammed aggregate piers should be a minimum of 24 inches in diameter and spaced 6 feet on center in a diamond pattern. A minimum of 3 rows of piers should be utilized.

Following installation of the piers, post construction testing utilizing CPT's and analyses should be performed by the Geotechnical Engineer to confirm that lateral spreading safety factor exceeds 1.5 for lateral spreading behind the mitigation zone.
In addition a minimum of 3 modulus tests at locations determined by the Geotechnical Engineer should be performed per ASTM 1143 and be shown to achieve a minimum modulus of 225 p.c.i.

BUILDING FOUNDATION SUPPORT

Engineered aggregate piers should also be used to provide support of the building foundations. To provide proper support of the foundations, the aggregate piers should extend at least 12 inches into the underlying bedrock. Within the building areas, the aggregate piers should be installed so that they extend 6 to 12 inches above the bottom of the footings so that when the footings are excavated, the upper portions of the piers are shaved off. This procedure allows positive identification of the piers by the project geotechnical consultant and building inspector at the time of the footing inspections.

Beyond the 20-foot-wide ground modification zone, the spacing of the aggregate piers will only be dependent on the loads of the building to be supported. To determine the gross number of piers required, it should be assumed that each aggregate pier can support approximately 80 kips of load. In addition, by installing anchor plates within the aggregate piers, uplift resistance can also be achieved. As an approximation, for a shaft length of 10 feet, an uplift capacity of 25 kips can be achieved. This shaft length is defined as the distance from the bottom of the spread footings to the contact with the bedrock. As the shaft length increases, a corresponding increase in uplift capacity will be achieved. Therefore, for a shaft length of 12 feet, an uplift capacity of 32.5 kips can be achieved while for a shaft length of 14 feet, an uplift capacity of 40 kips can be achieved. The spacing of the engineered aggregate piers should be determined by the project structural engineer based on their review of our soils report and laboratory tests and on the recommended bearing and uplift capacities.

The contractor should install the aggregate piers such that the bearing capacity and uplift pressures recommended previously are achieved and such that the total settlement does not exceed 1 inch. The adequacy of the aggregate piers to support the proposed buildings should be verified through the use of load and pullout tests performed on random aggregate piers. Additional aggregate pier installation may be required if the results of the load testing do not indicate an adequate level of foundation support or settlement amounts that are within tolerable limits. In addition a minimum of 3 modulus tests at locations determined by the Geotechnical Engineer should be performed per ASTM 1143 and be shown to achieve a minimum modulus of 225 p.c.i.

SITE PREPARATION AND GRADING RECOMMENDATIONS

GENERAL

The subject site should be precise graded in accordance with the City of Dana Point and County of Orange grading code requirements (and all other applicable codes and ordinances) and the recommendations as outlined in the following sections of this report. The geotechnical aspects of future grading plans and improvement plans should be reviewed by GMU Geotechnical prior to grading and construction. Particular care should be taken to confirm that all project plans conform to the recommendations provided in this report. All planned and corrective grading should also be monitored by GMU Geotechnical to verify general compliance with the recommendations outlined in this report.

DEMOLITION AND CLEARING

Prior to the start of the planned improvements, all materials associated with the existing buildings to be removed, including footings, floor slabs, and underground utilities, should be demolished and hauled from the site. The existing asphalt pavement sections, which are inadequate and severely damaged, will also need to be demolished. Due to the limited amount of grading and fill placement that will occur, the old asphalt and base materials generated from the removal of the existing pavement sections should be either recycled or collected and hauled off-site.

The on-site fill materials are suitable for use as new compacted fill from a geotechnical perspective if care is taken to remove all significant organic and other decomposable debris. Cavities and excavations created upon removal of subsurface obstructions, such as existing buried utilities, should be cleared of loose soil, shaped to provide access for backfilling and compaction equipment, and then backfilled with properly compacted fill.

The project geotechnical consultant should provide periodic observation and testing services during demolition operations to document compliance with the above recommendations. In addition, should unusual or adverse soil conditions or buried structures be encountered during grading that are not described herein, these conditions should be brought to the immediate attention of the project geotechnical consultant for corrective recommendations.

CORRECTIVE GRADING – BUILDINGS M1, 6 THROUGH 12, AND PARKING STRUCTURES (P-1 AND PODIUM P2)

The foundations of these new buildings and structures will be supported on engineered aggregate piers that extend into bedrock. However, to provide proper support of the building floor slabs, it is recommended the existing fill materials be overexcavated to a depth of at least 3 feet below proposed finish grades and these excavated materials be replaced as properly compacted fill placed at a minimum relative compaction of at least 90 percent.

CORRECTIVE GRADING – BUILDING 4 ADDITION

As described previously, we recommend that the foundations of the new buildings and structures proposed within the site be supported on engineered aggregate piers that extend into bedrock. However, there is not sufficient room to install engineered aggregate piers below the proposed addition to Building 4. Furthermore, since this small addition will exert relatively light loads on the existing soils, engineered aggregate piers are not considered necessary. However, in order to reduce the potential for future settlement, it is recommended the existing fill materials beneath this addition be overexcavated to a depth of at least 3 feet below proposed finish grades**, or to a depth of at least 2 feet below proposed footings, whichever is greater**, and these excavated materials then replaced as properly compacted fill placed at a minimum relative compaction of at least 90 percent.

CORRECTIVE GRADING – EXTERIOR PARKING, DRIVEWAY AND HARDSCAPE AREAS

It is expected that the existing surficial fill materials will be disturbed during the demolition of the existing asphalt pavement sections. Therefore, to provide adequate support of proposed exterior improvements such as parking lots and driveways, and hardscape features such as patios, walkways, stairways and planter walls, the existing ground surfaces in these areas should be overexcavated to a depth of at 2 feet below proposed finish grades and these excavated materials then replaced as properly compacted fill placed at a minimum relative compaction of at least 90 percent.

FILL MATERIAL AND PLACEMENT

Suitability

All on-site soils are considered suitable for use as compacted fill from a geotechnical perspective if care is taken to remove all significant organic and other decomposable debris, and separate and stockpile rock materials larger than 6 inches in maximum diameter.

Compaction Standard and Methodology

All soil material used as compacted fill, or material processed in-place or used to backfill trenches, should be moistened, dried, or blended as necessary and densified to at least 90% relative compaction as determined by ASTM Test Method D 1557. It is recommended that fills be placed a minimum of 2% above optimum moisture content.

Material Blending

The existing surficial engineered fill materials are expected to be generally slightly below optimum moisture content but may have variable moisture content depending on the season in which work is performed. The majority of the materials to be handled during grading will require some blending and addition of water to meet acceptable moisture ranges for sufficient compaction (i.e., minimum 2% above optimum moisture content).

Use of Rock or Broken Concrete

Significant rock materials greater than 6 inches in diameter are not anticipated during the subject grading. Due to the limited amount of grading and fill placement that will occur, any oversize rock materials generated during grading should be collected and hauled off-site.

TEMPORARY SLOPE STABILITY

During site grading, temporary laid back slopes up to approximately 5 to 6 feet in height are expected to be created during the construction of proposed retaining walls. The sidewalls of these temporary slopes are expected to expose existing artificial fill materials.

Based on the anticipated engineering characteristics of these materials, temporary slopes to a maximum height of 4 feet may be cut vertically without shoring subject to verification of safety by the contractor. Deeper excavations should be braced, shored, or those portions of the sidewalls above a height of 4 feet should be sloped back no steeper than 1:1 (horizontal to vertical). In addition, no surcharge loads should be allowed within 10 feet from the top of the temporary slopes.

We anticipate the slopes will be temporarily stable provided the above recommendations are followed. However, modifications to these recommendations may be required based on our observations of the actual conditions exposed in the field

Our temporary slope recommendations are provided only as general guidelines and all work associated with temporary slopes should meet the minimal requirements as set forth by CAL-OSHA. Temporary slope construction, maintenance, and safety are the responsibility of the contractor.

POST-GRADING AND GROUND IMPROVEMENT CONSIDERATIONS

UTILITY TRENCHES

Utility Trench Excavations

The subject site is underlain by approximately 15 to 20 feet of fill materials that are highly variable and consist of frequently alternating layers of clayey sands, silty sands, sands, sandy clays, and sandy silts. In general, the granular sand materials were found to be medium dense to dense while the fine grained clay and silt materials were found to be predominantly firm to very firm. Furthermore, groundwater was encountered at relatively shallow depths (8 to 14 feet and at 5 feet within one locality).

For this condition, the soils above the groundwater level can be considered as OSHA soil type C and should be laid back at a maximum slope ratio of 1.5:1, horizontal to vertical. In addition, surcharge loads should not be allowed within 10 feet of the top of the excavations.

For deeper trenches, groundwater will be encountered and the contractor should develop an approach for dewatering, shoring, and addressing shallow groundwater conditions. Sumping and pumping of free water from open excavations is not expected to result in dry and stable trench conditions due to the close proximity of the adjacent bay; therefore, a dewatering system will need to be designed, installed, and operated by an experienced company specializing in groundwater dewatering systems. The dewatering system should be capable of lowering the groundwater surface to a depth of 5 feet below the bottom of the trenches. Before implementing a dewatering system, we recommend that a dewatering test program be conducted to evaluate the feasibility and efficiency of the proposed dewatering system. Dewatering should be performed and confirmed by potholing or other means prior to trench excavation. Dewatering operations will also need to comply with all NPDES regulations.

Temporary shoring will be required below the water table, where saturated soils are encountered, or where vertical trench sidewalls are desired. Shoring should consist of metal, plywood, and/or timber sheeting supported by braces or shields. Trench walls lacking sheeting will be unstable and experience sloughing. Trench shields will only provide worker safety and will not provide full support of the trench walls unless the shields are installed tightly against the sidewalls. Lateral pressures considered applicable for the shoring design will depend on the type of shoring system selected by the contractor and whether the site is dewatered. GMU can provide specific design values once the type of shoring is determined.

The above recommendations are presented as guidelines only and are minimum requirements. Temporary trench excavation construction, maintenance, and safety are the responsibility of the pipeline contractor. The contractor should retain a qualified and experienced registered engineer to design any shoring systems in accordance with OSHA criteria. The shoring engineer should evaluate the adequacy of the shoring design parameters provided in this report and make appropriate modifications, as necessary. The design should consider local groundwater levels as reported herein and that groundwater levels may change over time as a result of tidal influences.

Utility Trench Subgrade Stabilization

Prior to pipeline bedding placement, the trench subgrades should be firm and unyielding. If unsuitable subgrade soils are encountered, the contractor should consult with the project geotechnical engineer to provide subgrade stabilization. Stabilization may generally consist of the placement of crushed rock or processed miscellaneous base. Crushed rock, if used, would need to be encased in filter fabric. Specific recommendations would be dependent on actual conditions encountered.

Utility Trench Backfill Considerations

Backfill compaction of utility trenches should be such that no significant settlement will occur. Backfill for all of these trenches should be compacted to at least 90% relative compaction subject to sufficient observation and testing. In the event that granular material having a sand equivalent of 30 or greater is used for backfill and this material is thoroughly flooded into place, extensive testing is not required. If native material with a sand equivalent less than 30 is used for backfill, it should be placed at near-optimum moisture content and mechanically compacted. Jetting or flooding will not densify native soil materials with a sand equivalent less than 30 due to their silty to clayey nature. Also, jetting or flooding of granular material should not be used to consolidate backfill in trenches adjacent to any foundation elements.

Where trenches closely parallel a footing (i.e., for retaining walls) and the trench bottom is located within a 1 horizontal to 1 vertical plane projected downward and outward from any structure footing, concrete slurry backfill should be utilized to backfill the portion of the trench below this plane. The use of concrete slurry is not required for backfill where a narrow trench crosses a footing at about right angles.

We suggest that these recommendations be included as a specification in all subcontracts for underground improvements. In addition, the design of all underground conduits, pipelines, or utilities should also consider the potentially corrosive nature of the on-site soils to metals, as previously described in this report.

SURFACE DRAINAGE

Surface drainage should be carefully controlled to prevent runoff over graded slope surfaces and ponding of water on flat pad areas. Positive drainage away from graded slopes is essential to reduce the potential for erosion or saturation of slope surfaces. Maintaining positive drainage of all landscaping areas along with avoiding over-irrigation will help minimize the possibility of "perched" groundwater accumulating slightly below the graded surfaces. All drainage at the site should be in minimum conformance with the applicable City of Dana Point codes and standards.

FOUNDATION DESIGN RECOMMENDATIONS

STRUCTURE SEISMIC DESIGN

No active or potentially active faults are known to cross the site; therefore, the potential for primary ground rupture due to faulting on-site is very low to negligible. However, the site will likely be subject to seismic shaking at some time in the future. For design of future buildings, retaining walls or other structural improvements, CBC seismic design parameters were determined using the USGS computer program titled "Seismic Hazard Curves and Uniform Hazard Response Spectra, Version 5.0.8." The site coordinates used in the analysis were 33.46085^{o} North Latitude and 117.69342 West Longitude. In addition, these parameters were determined assuming that the liquefaction potential of the onsite soils will be mitigated through ground modification as recommended previously. Based on these anticipated conditions, on-site structures should be designed in accordance with the following 2010 CBC criteria:

Parameter	Factor	Value
0.2s Period Spectral Response	S_s	1.608g
1.0s Period Spectral Response	S_1	0.589g
Soil Profile Type	Site Class	C
Site Coefficient	F_a	1.0
Site Coefficient	F_v	1.3
Adjusted Spectral Response	SM_s	1.608g
	SM_1	0.766g
Adjusted Spectral Response	SD_s	1.072g
	SD_1	0.511g

It should be recognized that much of southern California is subject to some level of damaging ground shaking as a result of movement along the major active (and potentially active) fault zones that characterize this region. Design utilizing the 2010 CBC is not meant to completely protect against damage or loss of function. Therefore, the preceding parameters should be considered as minimum design criteria.

If requested by the project structural engineer, GMU can also provide a site-specific ground motion hazard analysis per ASCE 7-05 Sections 21.2, 21.3, and 21.4.

GENERAL

The following preliminary foundation design recommendations are provided based on anticipated conditions at the completion of anticipated ground modification and grading; however, these recommendations are based on conceptual plans that may be revised during the plan check process. Ultimate construction and grading within the site should be in accordance with all applicable provisions of the grading and building codes of the City of Dana Point, the current 2010 CBC, and all of the recommendations of the project civil and geotechnical consultants involved in the final site development

DRY STORAGE BUILDING (BUILDING #M1)

This building is proposed in close proximity of the existing quay seawall with the southwestern end of the building actually extending out over the seawall and above the harbor water. Due to its proposed location alongside and out over the existing seawall, three different foundation systems will be required beneath this building.

1. The portion of the building that extends out over the water will be supported on piles that are driven or drilled through the surficial soils and socketed into the underlying bedrock.

2. To mitigate the potential for distress due to lateral spreading, the portion of the building located near the seawall will need to be supported on aggregate stone piers that have been designed to mitigate the potential for seismic-induced lateral spreading.

3. To mitigate the potential adverse effects of the settlement of the highly variable fill materials, the remainder of this building should also be supported on aggregate stone piers but they only need to provide bearing support. Recommendations for the installation of the aggregate piers were provided previously in the "Ground Modification Recommendations" section of this report.

To provide support of the building slab, it is recommended that the subgrade soils below the floor slabs be overexcavated and recompacted to a minimum depth of 3 feet as described in the "Site Preparation and Grading" section of this report. Design parameters for the foundations are provided below.

Geotechnical Design Parameters for Overwater Piles

Design parameters for piles to be used out over the water of the harbor are presented below with supporting calculations contained in Appendix E:

- Piles: 24-inch driven or drilled and grouted octagonal piles
- Allowable Pile Capacity: Per Sheet 10 of 11 of Appendix E (300 kip maximum)
- Minimum Pile Spacing: 3d (3 x pile diameter)
- Pile Lengths: Per Sheet 10 of 11 of Appendix E (15 feet minimum into bedrock)
- Maximum Down Drag Load: Not applicable
- Pile Group Efficiency: 1.0
- Pile Settlement: Less than 0.5”
- Lateral Pile Response: L-pile geo-material model provided on Sheet 11 of 11 of Appendix E (L-pile analysis to be performed by structural engineer)

Geotechnical Design Parameters for Spread Footings

The following recommendations for spread footings are provided based on the assumption that the portion of the building to be constructed on-grade will be underlain by engineered aggregate piers as recommended previously.

<u>Rammed Aggregate Piers</u> The configuration of the aggregate piers bearing in bedrock should be determined by the structural engineer assuming each pier has an allowable load carrying capacity of 80 kips.

<u>Minimum Footing Depth.</u> The minimum footing depth recommended for the spread footings of the proposed building is 24 inches below top of slab (interior footings) and lowest adjacent outside grade (for perimeter footings). Reinforcement should be determined by the structural engineer.

<u>Bearing Materials</u>. All foundations should bear onto engineered aggregate piers approved by a representative from GMU.

<u>Bearing Value</u>. An allowable bearing pressure of 6,000 pounds per square foot (psf) may be used for spread foundations at least 2 feet wide and embedded a minimum of 24 inches below the top of slab or lowest adjacent grade. These values may be increased by one-third for short term wind and seismic loads.

Lateral Load Design. Lateral loads may be resisted by friction at the base of the foundations and by passive resistance within the adjacent earth materials and aggregate piers. A coefficient of friction of 0.35 may be used between the foundations and the recommended bearing material. A passive resistance equal to 350 pounds per square foot per foot of embedment may be assumed to a maximum of 1,750 pounds per square feet. These values may be increased by one-third for short term wind and seismic loads. In addition, the upper 6 inches of embedment for the at-grade foundations should be disregarded when calculating passive pressures.

Post-Construction Movements (Settlement)

Settlement of the piles located out over the water is expected to be less than ½ of an inch. Provided that the remainder of the building is supported on engineered aggregate piers, settlement of the spread footings can be expected to be less than 1 inch under a bearing pressure of 6,000 pounds per square foot. Differential settlement can be estimated to be ½ of an inch over a span of 40 feet.

Slab Design

The portion of the building to be constructed out over the water of the bay will need to be designed with a structural slab that only derives support from the piles and grade beams. This structural slab should extend at least 15 feet to the northeast (i.e., inland) of the existing seawall.

The boat storage portion of the building to be constructed on-grade and with spread foundations may be constructed with a conventional slab; however, since this slab will need to support highly repetitive forklift and will be sensitive to any future settlement or distress, it should have a minimum thickness of at least 12 inches and be minimally reinforced with No. 4 bars at 18 inches on center.

The floor slabs for the office portion of the building should have a minimum thickness of 5 inches and be minimally reinforced with No. 4 bars at 18 inches on center while the floor slabs for the maintenance canopy should have a minimum thickness of 8 inches, be minimally reinforced with No. 4 bars at 18 inches on center, and be underlain by 6 inches of base.

Final determination of slab thickness and reinforcement should be determined by the structural engineer.

Subgrade Soil Moisture Content

The foundation subgrade should be moisture conditioned/pre-saturated as necessary to at least 2% over the optimum moisture content to a minimum depth of 18 inches. The moisture content of the subgrade soils should be verified by GMU prior to initiating foundation construction.

BUILDINGS 6-12, PARKING DECK P-1 AND PODIUM P2, AND GOLDEN LANTERN ENTRANCE RAMP

These buildings and structures will have a fairly significant range in foundation loads while at the same time will be interconnected by either the Podium structure or the pedestrian bridge. Therefore, differential vertical and lateral movements of these buildings and structures cannot be tolerated. As a result, those portions of the buildings and structures located near the seawalls will need to be supported on aggregate stone piers that are designed to eliminate the potential for liquefaction induced lateral spreading. To mitigate the potential for distress due to differential settlement, it is recommended that the remainder of these buildings and structures also be supported on aggregate stone piers. To provide support of the building slabs, it is recommended that the subgrade soils below the floor slabs be overexcavated and recompacted as described in the "Site Preparation and Grading" section of this report.

Geotechnical Design Parameters for Spread Footings

The following recommendations for spread footings are provided based on the assumption that the portion of the building to be constructed on-grade will be underlain by engineered aggregate piers as recommended previously.

<u>Rammed Aggregate Piers</u> The configuration of the aggregate piers bearing in bedrock should be determined by the structural engineer assuming each pier has an allowable load carrying capacity of 80 kips.

<u>Minimum Footing Depth.</u> The minimum footing depth recommended for the proposed buildings is 24 inches below top of slab (interior footings) and lowest adjacent outside grade (for perimeter footings). Reinforcement should be determined by the structural engineer.

Minimum Footing Setbacks. The footings for Building 12 which are located in close proximity of a descending slope should be further deepened, as necessary, such that they meet the setback requirements of both the County of Orange Grading Manual and Grading and Excavation Code and the 2010 CBC.

<u>Bearing Materials</u>. All foundations should bear onto engineered aggregate piers approved by a representative from GMU.

<u>Bearing Value</u>. An allowable bearing pressure of 6,000 pounds per square foot (psf) may be used for foundations at least 2 feet wide and embedded a minimum of 24 inches below the top of slab or lowest adjacent grade. These values may be increased by one-third for short term wind and seismic loads.

<u>Uplift</u>. An allowable bearing pressure of 6,000 pounds per square foot (psf) may be used for foundations at least 2 feet wide and embedded a minimum of 24 inches below the top of slab

or lowest adjacent grade. These values may be increased by one-third for short term wind and seismic loads.

Lateral Load Design. Lateral loads may be resisted by friction at the base of the foundations and by passive resistance within the adjacent earth materials and aggregate piers. A coefficient of friction of 0.35 may be used between the foundations and the recommended bearing material. A passive resistance equal to 350 pounds per square foot per foot of embedment may be assumed to a maximum value of 1,750 pounds per square feet. For the footings of Building 12 which are located adjacent to the descending slope, a reduced passive resistance of 150 pounds per square foot to a maximum value of 750 pounds per square foot should be used. These values may be increased by one-third for short term wind and seismic loads. In addition, the upper 6 inches of embedment for the at-grade foundations should be disregarded when calculating passive pressures.

Post-Construction Movements (Settlement)

Provided that these buildings are supported on engineered aggregate piers as recommended previously, they should be designed for a total settlement of up to 1 inch with a differential settlement of ½ an inch over a horizontal distance of 40 feet.

Slab Design

Provided that these buildings are supported on engineered aggregate piers and that remedial grading is performed as described previously, they may be constructed with conventional slabs. The slabs should have a minimum thickness of 5 inches and be minimally reinforced with No. 4 bars at 18 inches on center. Final determination of slab thickness and reinforcement should be determined by the structural engineer.

Subgrade Soil Moisture Content

The foundation subgrade should be moisture conditioned/pre-saturated as necessary to at least 2% over the optimum moisture content to a minimum depth of 18 inches. The moisture content of the subgrade soils should be verified by GMU prior to initiating foundation construction.

Building 12 and Entrance Ramp Retaining Walls

Recommendations are provided for the retaining walls for the entrance ramp and Building 12. These walls are assumed to be restrained or at-rest. Calculations to support the recommendations are contained in the attached Appendix F.

- Foundation: Spread footings on Rammed Aggregate Piers (See above spread footing parameters)
- Unit Weight Backfill: 125 pcf

- At-Rest Earth Pressure: 63 pcf (drained pressure).
- Seismic Earth Pressure: 17 pcf (triangular distribution).
- Traffic Loading Pressures: 80 psf (where applicable).
- Geologic Surcharge: None
- Backdrainage: A backdrainage system should be placed behind all retaining walls and drain to an appropriate approved drainage facility.
- Waterproofing: All walls should be waterproofed. Detailed waterproofing recommendations are beyond our purview.
- Backfill: On-site, relatively non-expansive soil materials may be used to backfill retaining walls. The backfill materials should be approved by the geotechnical consultant with respect to their characteristics prior to placement. All wall backfill should be should be moistened, dried, or blended as necessary to achieve a minimum of 2% over optimum moisture content, and compacted to at least 90% relative compaction as determined by ASTM Test Method D 1557.
- Control Joints Control/Construction Joints should be implemented and designed by structural engineer. As a minimum, control, construction joints should be provided at maximum intervals of 15 to 20 feet and at all angle points and other locations where differential movement is likely to occur.

<u>Ramp Retaining Walls - Construction Sequence Requirements</u>

The following construction sequence should be utilized to construct the ramp retaining walls;

- Construct backfill with 1:1 false slopes up to ramp grade
- Monitor settlements until primary settlement is complete.
- Construct rammed aggregate piers below wall foundations.
- Backfill walls.

ADDITION TO BUILDING 4

Since this small addition will exert relatively light loads on the existing soils, engineered aggregate piers are not considered necessary and only remedial grading will be required. The remedial grading should be performed as described in the "Site Preparation and Grading" section of this report

Geotechnical Design Parameters for Spread Footings

<u>Minimum Footing Depth.</u> The minimum footing depth recommended for this proposed addition is 24 inches below top of slab (interior footings) and lowest adjacent outside grade (for perimeter footings). Reinforcement should be determined by the structural engineer.

<u>Bearing Materials</u>. All foundations may bear into new compacted fill approved by a representative from GMU.

<u>Bearing Value</u>. An allowable bearing pressure of 2,000 pounds per square foot (psf) may be used for foundations at least 2 feet wide and embedded a minimum of 24 inches below the top of slab or lowest adjacent grade. These values may be increased by one-third for short term wind and seismic loads.

<u>Lateral Load Design</u>. Lateral loads may be resisted by friction at the base of the foundations and by passive resistance within the adjacent earth materials. A coefficient of friction of 0.30 may be used between the foundations and the recommended bearing material. Passive resistance equal to 250 pounds per square foot per foot of embedment may be assumed to a maximum of 1,250 pounds per square foot. These values may be increased by one-third for short term wind and seismic loads. In addition, the upper 6 inches of embedment for the at-grade foundations should be disregarded when calculating passive pressures.

<u>Subgrade Soil Moisture Content</u>. The foundation subgrade should be moisture conditioned/pre-saturated as necessary to at least 2% over the optimum moisture content to a minimum depth of 18 inches. The moisture content of the subgrade soils should be verified by GMU prior to initiating foundation construction.

Slab Design

Provided that remedial grading is performed as described previously, this building addition may be constructed with a conventional slab. The slabs should have a minimum thickness of 5 inches and be minimally reinforced with No. 4 bars at 18 inches on center. Final determination of slab thickness and reinforcement should be determined by the structural engineer

Post-Construction Movements (Settlement)

Although ground modification will not be performed below this building addition, relatively light loads will be exerted on the underlying soil. Therefore, this addition should be designed for a total settlement of up to ¾ of an inch with a differential settlement of ½ an inch over a horizontal distance of 20 feet.

MOISTURE VAPOR BARRIERS

Due to the existing shallow groundwater table, a vapor barrier equivalent to Stego 15 should be utilized. The barrier should be installed as follows:

- Below the slabs of all buildings with habitable areas or where moisture-sensitive floor coverings are proposed.
- Installed per manufacture's specifications as well as with all applicable recognized installation procedures such as ASTM E 1643-98.
- Joints between the sheets and the openings for utility piping should be lapped and taped. If the barrier is not continuously placed across footings/ribs, the barrier should, as a minimum, be lapped into the sides of the footing/rib trenches down to the bottom of the trench.
- Punctures in the vapor barrier should be repaired prior to concrete placement.
- Prior to placing the barrier, a minimum of 4 inches of ¾-inch graded rock should be placed over the subgrade. The need for sand and/or the amount of sand above the moisture vapor retarder should be specified by the structural engineer. The selection of sand above the retarder is not a geotechnical engineering issue and is hence outside our purview. If the structural engineer requires sand above the barrier, it should consist of 1 to 2 inches of clean sand with a minimum sand equivalent of 30.

WATER VAPOR TRANSMISSION

As discussed above, placement of a moisture vapor barrier below certain slab areas is recommended. This moisture vapor barrier recommendation is intended only to reduce moisture vapor transmissions from the soil beneath the concrete and is consistent with the current standard of the industry for construction in Southern California. It is not intended to provide a "waterproof" or "vapor proof" barrier or reduce vapor transmission from sources above the barrier. Sources above the barrier include any sand placed on top of the barrier (i.e., to be determined by the project structural designer) and from the concrete itself (i.e., vapor emitted during the curing process). The evaluation of water vapor from any source and its effect on any aspect of the proposed living space above the slab (i.e., floor covering applicability, mold growth, etc.) is outside our purview and the scope of this report.

FLOOR COVERINGS

Prior to the placement of flooring, the floor slabs should be properly cured and tested to verify that the water vapor transmission rate (WVTR) is compatible with the flooring requirements.

CONCRETE

Based on the previously and recently performed and laboratory testing, the onsite soil and bedrock materials have negligible moderate concentrations of sulfates per Section 1904.3 of the 2010 CBC. In addition, concrete will have a potential exposure to seawater. Consequently, we recommend that minimum Type V cement along with a maximum water/cement ratio of 0.50 be used for all structural foundations in contact with the onsite soils. This recommendation will serve to minimize the potential of water and/or vapor transmission through the concrete and minimize the potential for physical attack to concrete from non-sulfate based salts. In addition, wet curing of the concrete as described in ACI Publication 308 should be considered.

The aforementioned recommendations in regards to concrete are made from a soils perspective only. Final concrete mix design as well as any concrete testing is outside our purview. All applicable codes, ordinances, regulations, and guidelines should be followed in regard to designing a durable concrete with respect to the potential for detrimental exposure from the on-site soils and/or changes in the environment.

CORROSION PROTECTION OF METAL STRUCTURES

The results of the laboratory chemical tests performed on soil samples collected within and adjacent to the subject area indicate that the on-site soils are corrosive to severely corrosive to ferrous metals. Consequently, metal structures which will be in direct contact with the soil (i.e., underground metal conduits, pipelines, metal sign posts, metal door frames, etc.) and/or in close proximity to the soil (wrought iron fencing, etc.) may be subject to corrosion. The use of special coatings or cathodic protection around buried metal structures has been shown to be beneficial in reducing corrosion potential. The potential for corrosion of ferrous metal reinforcing elements embedded in structural concrete will be reduced by the use of the recommended maximum water/cement ratio for concrete.

The laboratory testing program performed for this project does not address the potential for corrosion to copper piping. In this regard, a corrosion engineer should be consulted to perform more detailed testing and develop appropriate mitigation measures (if necessary). Otherwise, the on-site soils should be considered corrosive to copper.
The above discussion is provided for general guidance in regards to the corrosiveness of the on-site soils to typical metal structures used for construction. Detailed corrosion testing and recommendations for protecting buried ferrous metal and/or copper elements is beyond our purview.

SITE WALL AND RETAINING WALL DESIGN CRITERIA

General

Exterior site retaining and screen walls are proposed within landscape and parking areas. The criteria contained in the following sections may be used for the design and construction of these walls.

Retaining Wall Design Parameters

Recommendations are provided for the site exterior retaining walls. Recommendations are provided for both cantilever and restrained walls. Calculations to support the recommendations are contained in the attached Appendix F.

- Foundation: Cantilever wall with spread footings.
- Footing Width: 24 inches minimum.
- Minimum Depth: 18 inches below lowest outside adjacent grade
- Minimum Footing Reinforcement : Four #4 bars; two at top and two at bottom of footing (footings to be continuous across openings such as footpath gates).
- Allowable Bearing Capacity: 2000 psf with a minimum embedment of 18 inches (may be increased 20% for each additional foot of width or embedment to a maximum of 3,000 psf).
- Bearing Material: At least a 2-foot-thick section of engineered fill.
- Coefficient of Friction: 0.30
- Unit Weight of Backfill: 125 pcf
- Passive Earth Pressure: 250 psf/ft of depth (disregard upper 6 inches).
- Static Lateral Earth Pressures: 63 pcf (At-Rest).
  42 pcf (Active).
- Seismic Earth Pressure: 17 pcf (triangular distribution).
- Traffic Loading Pressures: 80 psf (where applicable).

- Backdrainage: A backdrainage system should be placed behind all retaining walls and drain to an appropriate approved drainage facility.

- Waterproofing: All walls should be waterproofed. Detailed waterproofing recommendations are beyond our purview.

- Backfill: On-site, relatively non-expansive soil materials may be used to backfill retaining walls. The backfill materials should be approved by the geotechnical consultant with respect to their characteristics prior to placement. All wall backfill should be should be moistened, dried, or blended as necessary to achieve a minimum of 2% over optimum moisture content, and compacted to at least 90% relative compaction as determined by ASTM Test Method D 1557.

- Control Joints: Control/Construction Joints should be implemented and designed by structural engineer. As a minimum, control, construction joints should be provided at maximum intervals of 15 to 20 feet and at all angle points and other locations where differential movement is likely to occur.

Screen Walls

For standard screen walls on flat ground, footings should be a minimum of 24 inches deep below the lowest outside adjacent grade. Wall foundations should be reinforced with two #4 bars top and bottom, and joints in the wall should be placed at regular intervals on the order of 10 to 20 feet. The wall foundation shall be underlain by at least a 2-foot-thick section of engineered fill.

POLE FOUNDATIONS

Pole foundations will be required for the light bollards for the new parking area. As a minimum, the pole foundations should be at least 18 inches in diameter and at least 3 feet deep; however, the actual dimensions should be determined by the project structural engineer based on the following design parameters.

<u>Bearing Materials</u>. The pole foundations may bear into engineered fill approved by a representative from GMU.

<u>Bearing Values</u>. End-bearing capacity and skin friction may be combined to determine the allowable bearing capacities of the pole foundations. An allowable bearing pressure of 2000 pounds per square foot (psf) may be used for pole foundations at least 18 inches in diameter and embedded a minimum of 3 feet below the lowest adjacent grade. A value of 350 pounds per square foot may be used to determine the skin friction between the concrete and surrounding soil.

<u>Lateral Load Design</u>. Lateral loads may be resisted by friction at the base of the foundations and by passive resistance within the adjacent earth materials. A coefficient of friction of 0.30 may be used between the foundations and the recommended bearing material. For passive resistance, an allowable passive earth pressure of 250 pounds per foot of pile diameter per foot of depth into competent bearing material may be used; however, passive resistance should be ignored within the upper foot due to possible disturbance during drilling. The passive resistance may be assumed to be acting over an area equivalent to two pile diameters.

CONCRETE FLATWORK DESIGN

Thickness and Joint Spacing

To reduce the potential for unsightly cracking related to the effects of moderately expansive soils, concrete walkways and patios should be at least 4 inches thick and provided with construction joints or expansion joints every 5 feet or less. Concrete walkways and patios should be underlain by a 4-inch-thick layer of Class 2 crushed aggregate base (CAB), crushed miscellaneous base (CMB), or equivalent, or clean sand having a sand equivalent of at least 30, should then be placed on top of the soil subgrade, moisture conditioned to at least optimum moisture, and compacted to at least 90% relative compaction.

Reinforcement

Concrete walkways and patios should be reinforced with No. 3 bars spaced 18 inches on centers, both ways. The reinforcement should be positioned near the middle of the slabs by means of concrete chairs or brick. Reinforcing bars should be provided across all joints to mitigate differential vertical movement of the slab sections. Walkways and patios should also be dowelled into adjacent curbs using 9-inch speed dowels with No. 3 bars or ½-inch steel or fiberglass bars at 18 inches on centers. If doweling is not performed, differential movement should be anticipated.

Subgrade Preparation

As a further measure to mitigate cracking and/or shifting of concrete flatwork, the subgrade soils below concrete walkways and patios should be compacted to a minimum relative compaction of 90 percent and then thoroughly watered to achieve a moisture content that is at least 2% over optimum. This moisture content should extend to a depth of approximately 12 inches into the subgrade soils and be maintained in the subgrade during concrete placement to promote uniform curing of the concrete and minimize the development of unsightly shrinkage cracks. Flooding or ponding of the subgrade is not considered feasible to achieve the above moisture conditions since this method would likely require construction of numerous earth berms to contain the water. Therefore, moisture conditioning should be achieved with sprinklers or a light spray applied to the subgrade over a period of several days just prior to pouring concrete. Soil density and presoaking should be observed, tested, and accepted by GMU prior to pouring the concrete.

All concrete has a tendency to crack and cracks in concrete can be caused by many different factors. When constructing concrete decks, patios, walkways, etc., it is important that the ground on which these improvements are to rest be properly prepared, including moisture conditioning. Slab thickness, location of joints, reinforcement, and concrete mixture must also be appropriate for the intended use. Proper placement, finishing, and curing of concrete are also very important factors in minimizing cracking.

PAVEMENT DESIGN CONSIDERATIONS

GENERAL

It is expected that the parking lots in Planning Areas 1 and 2, the streets, and the driveways within the site will be constructed with both asphalt pavement and Portland cement concrete. Therefore, recommendations for both types of pavement are provided in the following sections. In order to accommodate fire-truck and trash truck loading, a traffic index (T.I.) of 5.5 has been assumed for the drive areas, whereas a T.I. of 4.0 has been assumed for the parking stall areas. T.I. values for Casitas Place, Street of the Golden Lantern, and Puerto Place were provided to us by Mr. Michael Kennedy of Fehr and Peers. While the parking lot in Planning Area 2 will be used for conventional parking stalls and drive aisles for passenger vehicle and service vehicle access, Planning Area 1 parking areas will be subject to higher repeated traffic loading due to loading and unloading of boats along with usage by heavy loaded service vehicles. Thus we recommend that a T.I. of 5.5 be utilized for the design of the entire pavement section in the Planning Area 1 parking areas.

Several R-value tests were previously performed by others and during our recent geotechnical subsurface investigation, we obtained several other R-value tests results from other areas of the site. The results of all of these previous and current R-value tests ranged from 5 to 48. For design purposes, we recommend utilizing an R-value of 10 which will need to be confirmed during specific grading activities in each pavement area of the site.

ASPHALT PAVEMENT DESIGN

Based on an anticipated R-value of 10 to be obtained after precise grading of pavement subgrade areas, the following pavement thicknesses should be anticipated:

Location	R-Value	Traffic Index	Asphalt Concrete (in.)	Aggregate Base (in.)
Car Parking Stalls	10	4.0	3.0	6.0
Drive Aisles	10	5.5	4.0	9.5
Casitas Place	10	8.0	6.0	15.5
Street of the Golden Lantern	10	9.0	6.0	19.5
Puerto Place	10	8.0	6.0	15.5

Asphalt pavement structural sections should consist of crushed miscellaneous base (CMB) or crushed aggregate base materials (CAB) and asphalt concrete materials (AC) of a type meeting the minimum County of Orange requirements. The subgrade soils should be moisture conditioned to a minimum 2% above the optimum moisture content to a depth of at least 6 inches, and compacted to at least 90% relative compaction (per ASTM 1557). The CMB or CAB and AC should be compacted to at least 95% relative compaction (per ASTM 1557).

CONCRETE PAVEMENT DESIGN

Driveways and appurtenant concrete paving, such as trash receptacle bays, will require Portland cement concrete (PCC) pavement. Assuming a T.I. of 6 to 7, a design section of 8 inches of PCC over 6 inches aggregate base (AB) should be adequate. The AB should be Class 2 compacted to a minimum of 95% relative compaction as per ASTM D 1557.

FULL DEPTH RECLAMATION (FDR) ALTERNATIVE PAVEMENT FOR PLANNING AREA 1 PARKING AREAS

Since minor grade changes are planned for the re-grading of the Planning Area 1 parking areas, and based on site conditions and our experience we believe the most efficient pavement rehabilitation alternative to replacement with a conventional asphalt over base pavement section would be to utilize what is called “full depth reclamation” (FDR) utilizing a 12-inch-thick section of site reclaimed on-site AC and AB mixed with 6% cement to provide the new base for a new 4-inch-thick AC layer to be paved on top.
FDR has significant advantages over conventional pavement sections including the following major points:

- Savings in up-front costs (reusing materials, less excavation and import).
- Increased strength for weak in-place soils and long term service life (20 to 30 year design life).
- Reduced truck traffic to import and export materials.
- Environmental benefits and reduced community construction impact.
- Cautionary measures should be taken to avoid damaging existing utilities to ensure clearance for removal depths.

FDR can be performed in a similar construction schedule as presented below:

- Day 1 – Mill existing 1-inch top AC pavement surface and export. Light traffic can still drive on remaining AC section.
- Day 2 – Pulverize remaining AC and AB plus several inches of soil subgrade for a total of 12-inches of pulverization, mix in 6% Portland cement, moisture condition, and then compact to 95% relative compaction. Light traffic can drive on the FDR base layer at the end of the same day typically. Heavy truck traffic will be restricted.

- Day 3 – Curing FDR base layer. Closed to heavy truck traffic but light traffic can typically drive on FDR base.
- Day 4 – Micro crack FDR place base 4-inch-thick conventional Hot Mix Asphalt (HMA) AC layer and compact to 95% relative compaction. Light traffic can drive on base pavement section at the end of the same day.
- Day 5 – Place final HMA AC cap layer and compact.
- Day 6 - Heavier truck traffic can now be placed on new pavement section.

PERMEABLE INTERLOCKING CONCRETE PAVEMENT (PICP)

We understand that Permeable Interlocking Concrete Pavement (PICP) in the designated parking areas of Planning Area 1 may utilize a permeable interlocking concrete pavers such as “Eco-Stone”) and will assume subgrade soil conditions (R-value of at least 10) according to the “Design Manual for Permeable Interlocking Concrete Pavements” by ICPI (2011). The structural base thickness will need to be designed by the project civil engineer in order to meet storage requirements. This minimum section assumes a T.I. of up to 6.3 (GMU assumes a T.I. of 5.5 for the mixed use of parking and drives in this parking lot) and calls for a 3 1/8” (80 mm) concrete paver, over compacted layers of 2” of bedding course sand (ASTM No. 8 aggregate), over 4” of ASTM No. 57 stone as open-graded base, over 6” of ASTM No. 2 stone as open-graded sub base, over a Class 1 geotextile fabric* (highest strength) per AASHTO M-288.

*Due to the presence of gravel and some rock in the existing fill soils that will likely function as subgrade support for the PICP, GMU recommends using a Class 1 geotextile fabric (highest strength) placed both vertically at the sides of all PICP excavations and on top of the compacted subgrade soil below the stone sub base layer in order to protect the bottom and sides of the open-graded base and sub base. This geotextile fabric must meet AASHTO M-288 Class 1 geotextile strength property and subsurface drainage requirements (see attached Table 3-3 and Table 3-4 from page 31 of the ICPI Design Manual (2011) for AASHTO M-288 requirements).

CONCRETE INTERLOCKING VEHICULAR AND PEDESTRIAN PAVERS

We understand that portions of the project site will utilize 3 1/8-inch-thick (80 mm.) vehicular concrete interlocking pavers placed on a section of at least 1-inch-thick bedding sand. These vehicular pavers are also planned as a part of the subject project in order to provide fire department vehicle access capable of supporting 72,000 pounds of imposed loading. GMU recommends that the on-site soil subgrade in these site vehicular areas be scarified to a depth of 6 inches, moisture conditioned to at least 2% above the optimum moisture content, and compacted to at least 90% relative compaction. A geotextile fabric such as Mirafi 600X or equivalent should be placed on top of the compacted subgrade across the entire vehicular interlocking paver area. Based upon the on-site soils having an estimated R-value of 10, a 15-inch-thick layer of Class 2 crushed aggregate base (CAB), crushed miscellaneous base (CMB), or equivalent should be moisture conditioned to at least optimum moisture and compacted to at least 95% relative compaction in order to support the interlocking pavers. Concrete bands adjacent to the vehicular interlocking pavers should consist of a design section of 8 inches of PCC over at least 6 inches of AB or equivalent, moisture conditioned to at least optimum moisture, and compacted to at least 95% relative compaction.

We further understand that in certain designated site pedestrian areas, 2 3/8-inch-thick (60 mm.) concrete interlocking pavers placed on a section of at least 1-inch-thick bedding sand are planned. GMU recommends that prior to the installation of the pavers and bedding sand in these pedestrian areas, the on-site soil subgrade should be scarified to a depth of six inches, moisture conditioned to at least 2% above the optimum moisture content, and compacted to at least 90% relative compaction. A 4-inch-thick layer of Class 2 crushed aggregate base (CAB), crushed miscellaneous base (CMB), or equivalent should then be placed on top of the soil subgrade, moisture conditioned to at least optimum moisture, and compacted to at least 95% relative compaction in order to support the interlocking pavers in these pedestrian areas.

PLAN REVIEW/ GEOTECHNICAL TESTING AND OBSERVATIONS DURING CONSTRUCTION/ FUTURE REPORTS

Plan Review

Our office should review all future grading, foundation, and shoring plans for the site.

Geotechnical Observation and Testing

It is recommended that geotechnical observation and testing be performed by this firm during the following stages of construction and precise grading:

- During site clearing and grubbing.
- During removal of any buried lines or other subsurface structures.
- During all phases of excavation.
- During shoring installation.
- During installation of foundation and floor slab elements.
- During all phases of corrective, ground improvement, and precise grading including removals, scarification, ground improvement and preparation, moisture conditioning, proof-rolling, over-excavation, FDR treatment, and placement and compaction of all fill materials.
- During backfill of structure walls and underground utilities.
- During pavement and hardscape section placement and compaction.
- When any unusual conditions are encountered.

Future Reports

GMU should perform geotechnical reviews and provide geotechnical response letters to support the permit process for the grading, shoring, and building department reviews to support this report. The final project precise grading plans, and foundation plans for the project should also be reviewed by our office. In addition, geotechnical observation reports will be required following construction and grading.

LIMITATIONS

All parties reviewing or utilizing this report should recognize that the findings, conclusions, and recommendations presented represent the results of our professional geological and geotechnical engineering efforts and judgements. Due to the inexact nature of the state of the art of these professions and the possible occurrence of undetected variables in subsurface conditions, we cannot guarantee that the conditions actually encountered during grading and foundation installation will be identical to those observed and sampled during our study or that there are no unknown subsurface conditions which could have an adverse effect on the use of the property. We have exercised a degree of care comparable to the standard of practice presently maintained by other professionals in the fields of geotechnical engineering and engineering geology, and believe that our findings present a reasonably representative description of geotechnical conditions and their probable influence on the grading and use of the property.

Because our conclusions and recommendations are based on a limited amount of current and previous geotechnical exploration and analysis, all parties should recognize the need for possible revisions to our conclusions and recommendations during grading of the project. Additionally, our conclusions and recommendations are based on the assumption that our firm will act as the geotechnical engineer of record during precise grading and construction of the project to observe the actual conditions exposed, to verify our design concepts and the grading contractor's general compliance with the project geotechnical specifications, and to provide our revised conclusions and recommendations should subsurface conditions differ significantly from those used as the basis for our conclusions and recommendations presented in this report. It should be further noted that the recommendations presented herein are intended solely to minimize the effects of post-construction soil movements. Consequently, minor cracking and/or distortion of all on-site improvements should be anticipated.

The following services are outside our purview:

- Detailed corrosion testing and recommendations for protecting buried ferrous metal and/or copper elements.
- Environmental testing and/or evaluation of any kind.

SUPPORTING DATA/GRAPHICS

The following Plates and Appendices A through F which complete this report are listed in the Table of Contents.

Respectfully submitted,

GMU GEOTECHNICAL, INC.

David R. Atkinson

David R. Atkinson
Senior Engineer/Project Manager

PROFESSIONAL GEOLOGIST
LISA L. BATES
No. 2293
CERTIFIED ENGINEERING GEOLOGIST
Exp. 06-30-14
STATE OF CALIFORNIA

Lisa L. Bates, CEG 2293
Senior Engineering Geologist

REGISTERED PROFESSIONAL ENGINEER
GREGORY P. SILVER
GE 2336
Exp. Dec. 31, 2013
GEOTECHNICAL
STATE OF CALIFORNIA

Gregory P. Silver, M.Sc., PE, GE 2336
President/Principal Geotechnical Engineer

dwh/11-161-00R (7-19-13)

SITE-SPECIFIC REFERENCES

Bluewater Design Group, 2003, "Dana Point Marina Redevelopment, Bulkhead Structural Evaluation," December 17, 2003.

Diaz-Yourman and Associates, 2005, "Geotechnical Investigation, Dana Point Harbor Launch Ramp Rehabilitation, Dana Point, California." November 29, 2005, Proj. No. 2005-022.

GeoPentech, 2006, "Preliminary Geologic/Geotechnical Assessment Report for the Dana Point Harbor Revitalization Project Environmental Impact Report, January 14, 2004 (updated January 18, 2006).

Leighton Consulting Inc., 2002, "Preliminary Geotechnical Investigation Report for the Dana Point Harbor Master Plan, Dana Point Harbor, City of Dana Point, California." December 12, 2002, Proj. No. 600024-001.

Leighton Consulting Inc., 2008, "Geotechnical Engineering Exploration and Analysis for the Proposed Renovations to Dana Point Harbor, City of Dana Point, California," January 7, 2008, prepared for URS/Cash and Associates, Proj. No. 600024-004.

Zeiser Geotechnical, Inc., 1990, "City of Dana Point General Plan, Coastal Erosion Technical Report." PN 89312-2.

AERIAL PHOTOGRAPHS

DATE	FLIGHT	PHOTO
4-19-99	C136-45	170-171
10-15-97	C117-45	118-119
1-2-95	Cl01-45	10-11
1-14-92	C85-18	2-3
1-9-92	C-7	112-113-114
11-14-87	C-1	0012-0013
1-9-87	F	294-295
5-18-83	218-11	32-33
1-31-81	211-11	24-25
2-26-80	80033	268-269
12-13-78	203-11	43-44
1-24-77	181-11	31
1-13-75	157-11	27-28
10-29-73	132-10	20-21
1-31-70	61-10	223-224-225
3-30-67	2	94-95-96
9-20-65	1FF	86-87
3-28-59	261-R25	77-78
12-12-52	3K	49-50

TECHNICAL REFERENCES

Boore, D.M. and Atkinson, G.M., 2008, Ground-Motion Prediction Equations for the Average Horizontal Component of PGA, PGV, and 5%-Damped PSA at Spectral Periods between 0.01 s and 10 s: Earthquake Spectra, Vol. 24, No. 1, p. 99-138.

California Division of Mines and Geology, 1974, "Geology of the Dana Point Quadrangle, Orange County, California, Special Report 109," by Edgington, W.J.

California Division of Mines and Geology, 2001, "Seismic Hazard Zone Report for the Dana Point 7.5-Minute Quadrangle, Orange County, California," CDMG Seismic Hazard Zone Report 049.

California Division of Mines and Geology, 2008, "Guidelines for Evaluating and Mitigating Seismic Hazards in California," CDMG Special Publication 117, 76pp.

Campbell, K.W., and Bozorgnia, Y., 2008, NGA Ground Motion Model for the Geometric Mean Horizontal Component of PGA, PGV, PGD, and 5% Damped Linear Elastic Response Spectra for Periods Ranging from 0.01 to 10s: Earthquake Spectra, Vol. 24, No. 1, p. 139-171.

Chiou, B.S., and Youngs, R.R., 2008, An NGA Model for the Average Horizontal Component of Peak Ground Motion and Response Spectra: Earthquake Spectra, Vol. 24, No. 1, p. 173-215.

SCEC, 1999, "Recommended Procedure for Implementation of DMG Special Publication 117 Guidelines for Analyzing and Mitigating Liquefaction in California," Martin, G.R. and Lew, M., March 1999, 63 pp.