SOILS ENGINEERING, INC.
Consulting Geotechnical Engineers

REPORT OF GEOTECHNICAL INVESTIGATION

Proposed Monarch Bay Community Entrance and Park Improvements

Monarch Bay Drive and Pacific Coast Highway

City of Dana Point, California

Prepared For:

Monarch Bay Association

c/o Keystone Pacific Property Management, LLC.

16775 Von Karman Avenue, Suite 100

Irvine, California 92606

Project No. 6925.20 **April 20,2020**

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SOILS ENGINEERING, INC.
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April 20, 2020
Project No. 6925.20

Monarch Bay Association
c/o Keystone Pacific Property Management, LLC.
16775 Von Karman Avenue, Suite 100
Irvine, California 92606

Attention: Ms. Elizabeth Reed

Subject: **REPORT OF GEOTECHNICAL INVESTIGATION**
Proposed Monarch Bay Community Entrance and Park Improvements
Monarch Bay Drive and Pacific Coast Highway
City of Dana Point, California

Ladies and Gentlemen:

Presented herewith is the Report of Geotechnical Investigation (the Soils Report) prepared by Associated Soils Engineering, Inc. (ASE) for the proposed new community entrance and park improvements (the Improvements) at the subject location in the City of Dana Point, California (the Site). This work was conducted in accordance with ASE's Proposal No. P20-028, dated February 17, 2020, which subsequently received your authorization.

The subject geotechnical investigation was planned and performed based on relevant development information provided by your office. Provided information included a Park Preliminary Landscape Plan with Pool Option (Sheet 05), Community Entrance Preliminary Landscape Plan, and Preliminary Wall Plan and Elevations, prepared by Summers, Murphy & Partners, Inc. dated December 3, 2019.

The purpose of this study was to evaluate the subsurface soils conditions at the Site, followed by assessment of site geologic/seismic hazards, performance of engineering analyses, and formulation/assembly of recommendations for the geotechnical design and construction pertinent to the Improvements. ASE's study has concluded that construction of the Improvements is geotechnically feasible provided that the recommendations and criteria with respect to site grading and foundation design/construction presented in the Soils Report are incorporated in the project plans and implemented during construction. This Soils Report also presents 1) the findings of the geotechnical field investigation, 2) the summary of potential geological/seismic hazard assessment, and 3) the results of laboratory tests performed.

We at ASE appreciate the opportunity to provide our professional services on this important project and look forward to assisting you during construction phase of the Improvements.

If you have any questions or require additional information, please contact the undersigned.

Respectfully submitted,
ASSOCIATED SOILS ENGINEERING, INC.

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1.0 INTRODUCTION

This Soils Report presents the results of ASE's geotechnical investigation for the proposed new community/park improvements (the Improvements) within the Monarch Bay Community located on the southwest side of Pacific Coast Highway and Monarch Bay Drive, in the City of Dana Point, California (the Site). The approximate location of the Site is shown on the Site Location Map (Figure 1). The purpose of this investigation was to evaluate the general subsurface soil conditions at the Site and provide geotechnical recommendations for the design and construction of the Improvements. This Soils Report presents the summary of ASE's field findings, laboratory test data, and the results of engineering evaluations/analyses, which form the basis for formulating pertinent geotechnical conclusions and recommendations.

1.1 Project Outline

ASE understands that the following provided project information is applicable at the time of preparation of this Soils Report.

1.1.1 Building/Development Scope:

ASE understands that the project is to consist of the removal of the existing guardhouse, park building and improvements and construction of entirely new Improvements. The Improvements are to include a new guardhouse, multi-use building, bathroom, sunken tennis court, potential pool and spa, reconfigured entry/parking area, and new paved entry drive. The Improvements are to also include a new split face block wall (462 linear feet) parallel to Pacific Coast Highway, south of the entrance. The guardhouse, bathroom and multi-use building (the Buildings) have been assumed to be of single-story wood frame, stucco and masonry construction, with conventional footings. The finish grades will be near existing site grades (± one foot). Appurtenant construction will likely include the associated utility connections, pavements, landscaping and hardscaping, and site walls.

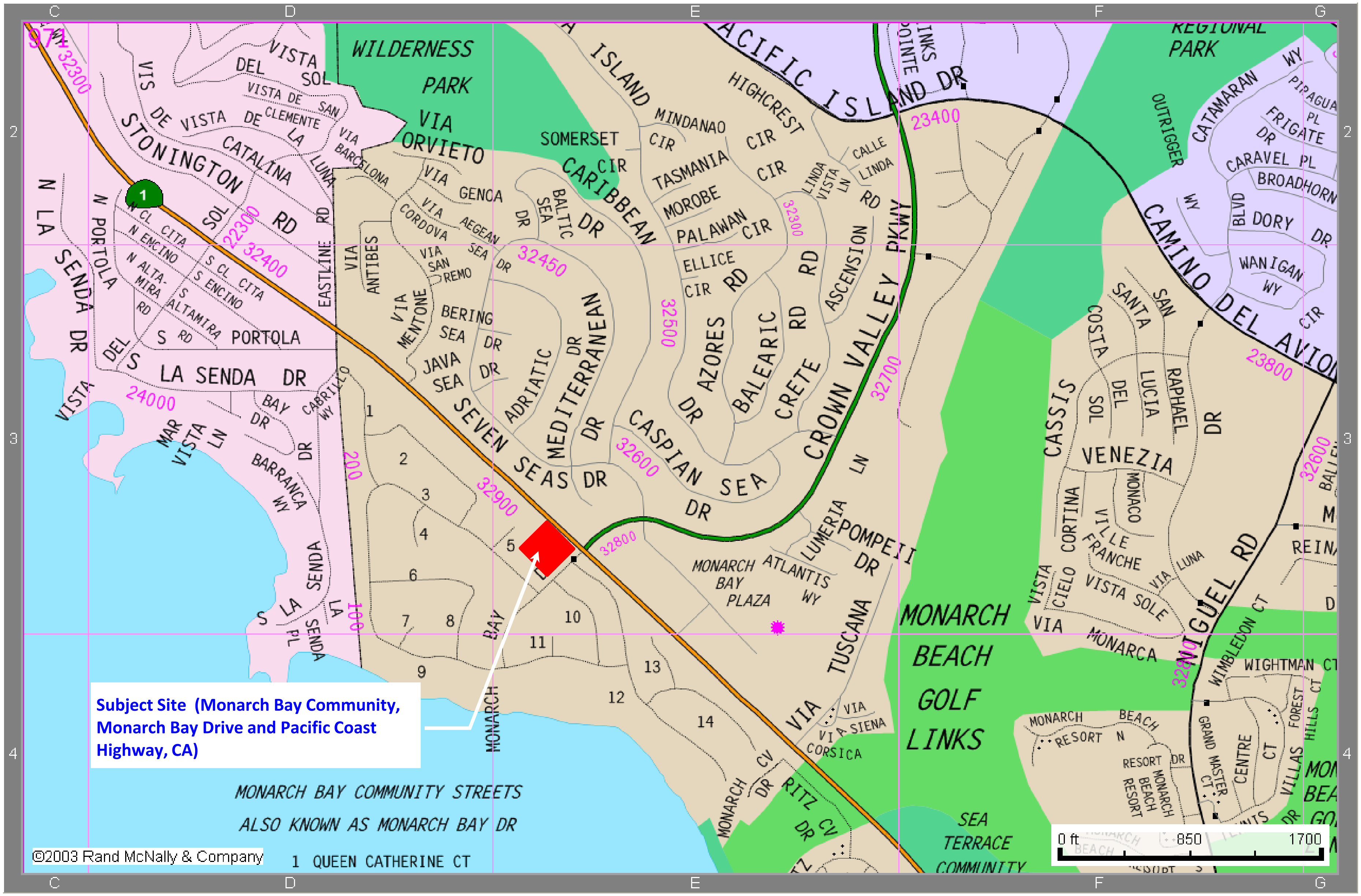
1.1.2 Structural Loading for Geotechnical Analyses:

For geotechnical evaluation purposes, ASE has assumed that the Improvements will be supported by isolated pad footings with maximum concentrated column load (D + L) on the order of 25 kips, and by continuous spread footings with maximum line load (D + L) of approximately 2,500 pounds per linear foot. Tolerable total and differential settlements resulted from the aforementioned structural loadings on the order of one (1) inch and 1/4 inch over a 30-foot distance, respectively, have also been assumed by ASE.

1.2 Scope of Exploration

In accomplishing the subject investigation, ASE's staff had performed the following geotechnical tasks:

A. Review of readily available background information, including in-house geotechnical data, geotechnical literature, geologic maps, seismic hazard maps, and literature relevant to the Site.



Associated Soils Engineering, Inc.	Site Location Map	Proj. Name:	Monarch Bay Community/Park Improvements, Monarch Bay Drive and PCH, Dana Point
2860 Walnut Avenue			
Signal Hill, CA 90755	Figure 1	Proj. No.:	6925.20
Tel (562) 426-7990 Fax (562) 426-1842		Date:	April, 2020

ASSOCIATED SOILS ENGINEERING, INC.
Consulting Geotechnical Engineers

B. A site reconnaissance to observe the existing Site conditions and to select/mark boring locations, followed by 72-hour advance notification to Underground Service Alert of the planned investigation.

C. Field exploration consisting of drilling five (5) exploratory borings to depths ranging from 2 feet to 14 feet 10 inches below respective existing grades. ASE staff logged and sampled representative soils encountered in each exploratory boring. Locations of the exploratory borings on site are shown on the Boring Location Plan, Plate A, in Appendix A.

D. Laboratory testing on retrieved representative soil samples for classification and for determination of pertinent engineering properties.

E. Engineering analyses of data obtained from literature review, the site, and laboratory testing, covering the following aspects:

   - Evaluation of general subsurface conditions and description of types, distribution, and engineering characteristics of subsurface materials.
   - Assessment of geologic/seismic hazards based on the updated criteria of the California Geological Survey (CGS).
   - Determination of the seismic design parameters in accordance with Chapters 16 and 18 of the California Building Code, 2019 Edition (2019 CBC).
   - Evaluation of the suitability of on-site soils for foundation support and establishment of qualification criteria of fill material, covering both on-site and imported soils.
   - Recommendations for site remedial grading and subgrade preparation.
   - Recommendations for design of footing foundations including minimum dimensions, allowable bearing capacity, estimated settlement, and lateral resistance.
   - Recommendations for slab-on-grade, covering subgrade preparation, design criteria, and construction guidelines.
   - Recommendations for temporary excavation and shoring.
   - Recommendations for asphaltic concrete (AC) and Portland Cement Concrete (PCC) pavements.
   - Recommendations for interlocking concrete pavers.
   - Recommendations for pool design.
   - Evaluation of the corrosion and expansion potential of the on-site materials.

F. Preparation of this Soils Report presenting the work performed and data acquired, as well as summarizing the conclusions and geotechnical recommendations for various aspects of design and construction with regard to the Improvements.

Please note that ASE's geotechnical investigation did not include any evaluation or assessment of hazardous or toxic materials which may or may not exist on or beneath the site. ASE does not consult in the field of potential site contamination/mitigation.

2.0 SITE AND SUBSURFACE CONDITIONS

2.1 Location, Boundary Conditions and Existing Development

The Improvements are to be located within the Monarch Bay private community, on the southwest side of Monarch Bay Drive and Pacific Coast Highway, in the City of Dana Point.

The Site is bound to the northeast by Pacific Coast Highway, with existing single-family residential development beyond to the north, and commercial development beyond to the east. The Site is bound to the south and west by existing single-family residential development.

The Site is presently occupied by existing community/park improvements, including existing lawn, guardhouse, multi-use building, tennis court, basketball court and parking lot. The area of the Buildings is generally level, but not uniform, with a surface gradient towards the southeast. The paved drive adjacent to the location of the proposed block wall slopes down to the southeast (approximately 30 feet lower than park grades). A small electrical substation is adjacent to Pacific Coast highway northwest of 311 Monarch Bay Drive. A large banyan tree is located at the west corner of Monarch Bay Drive and Pacific Coast Highway. Hedges are along the Site perimeters.

2.2 Subsurface Conditions and Geology

2.2.1 Artificial Fill (af):

Artificial fill was not observed in any of ASE’s exploratory borings, but may be present at other areas of the Site, or could be encountered during site grading, subject to the observation and confirmation of the Geotechnical Consultant.

2.2.2 Late to Middle Pleistocene-Age Old Marine Deposits (Qom):

Native site soils consisting of late to middle Pleistocene-age marine deposits were encountered in ASE’s borings. Per Reference 4, the older marine terrace deposits occur above marine wave-cut platforms located on top of the coastal bluffs. Soils within the unit were found to predominantly consist of sand, clay and silt. In specific, on-site alluvial soils consist of silty sands, silty clay, silty clays with sand, sands, clayey sands, and sands with silt. The granular, sandy strata of the site native soils are in a medium dense to dense condition, whereas the finer-grained, cohesive strata (i.e. clays) generally exhibit firm to stiff consistencies. Site subsurface soils were, in general, in a dry to moist condition within the respective explored depths at the time of ASE’s site investigation.

Figure 2, Local Geologic Map, excerpt from Reference 4, shows geologic material distribution in the vicinity of the Site.

More detailed descriptions of soils encountered and conditions observed during the subsurface exploration are shown in the Field Logs of Borings ("B" Plates) in Appendix A, together with information including soil classifications, depths and types of soil samples, field dry densities and moisture contents, and corresponding laboratory tests performed. Please note that the subsurface soils descriptions presented above have been interpreted from conditions exposed during the field investigation and/or information inferred from the reviewed geologic literature. As such, it is likely that not all of the subsurface conditions at the Site could be captured or represented. It is therefore essential that the Geotechnical Consultant's engineer or geologist be on site during grading and foundation construction such that information/recommendations deciphered during preliminary geotechnical investigation phase could be verified and, if necessary, amended as appropriate.

2.3 Groundwater and Caving

During field exploration, groundwater was not encountered to the maximum explored depth of 14 feet 10 inches below grade in Boring B-2. A search on Google Earth indicates that the Site elevation at the park site is approximately 158 feet above Mean Sea Level (MSL).

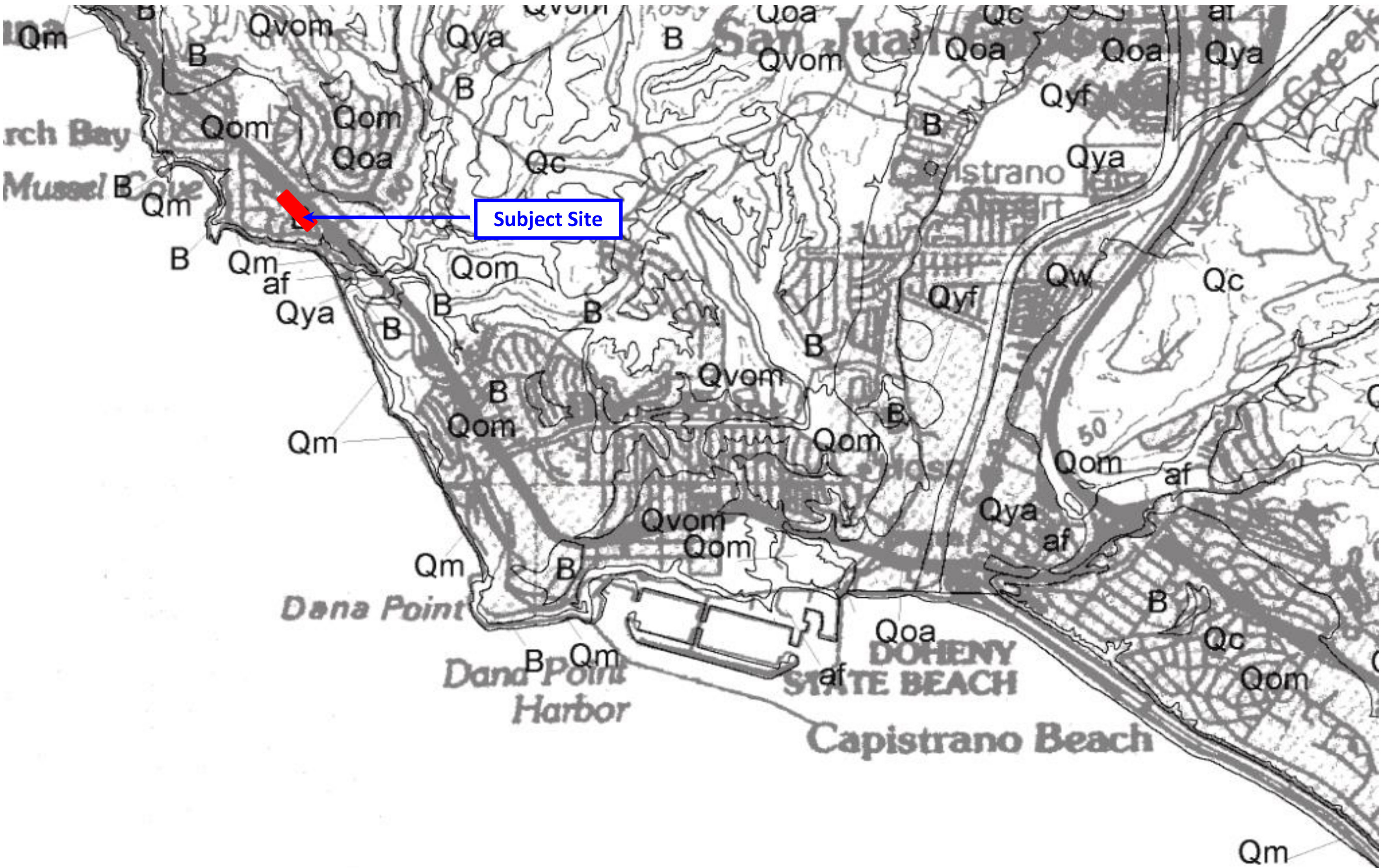
Information from the State of California Water Resources Control Board Geotracker website (http://geotracker.waterboards.ca.gov) indicates that the historic high groundwater level in groundwater monitoring well B-2R, located near the north corner of Pacific Coast Highway and Vista Del Sol (Shell Oil T0605902607: 32342 Pacific Coast Highway - approximately 0.65 mile northwest of the Site), was 68.61 feet below well surface elevation on September 5, 2018. The well surface elevation is 188 feet above MSL, or 30 feet higher than Site grade.

Generally, seasonal and long-term fluctuations in the groundwater may occur as a result of variations in subsurface conditions, rainfall, run-off conditions and other factors. Therefore, variations in groundwater levels from the short-term observations made in ASE's exploratory borings cannot be ruled out. Please note that ASE's exploratory borings were not meant for groundwater monitoring.

Caving and/or sloughing were not measured during the performance of excavation and sampling operations performed with manual equipment. However, caving and/or soil sloughing cannot be ruled out in excavations greater in dimension than our exploratory borings.

2.4 Utilities

No overhead or underground utilities were encountered or disturbed during the course of ASE's on-site exploration. However, underground utilities provide service to the existing guardhouse, multi-use building,



Partial extract of Quaternary Geologic Map of the Dana Point 7.5-Minute Quadrangle, California; California Geological Survey, SHZR 049, 2001

LEGEND			
■ Approximate Site Location Qom	Associated Soils Engineering, Inc. 2860 Walnut Avenue Signal Hill, CA 90755 Tel (562) 426-7990 Fax (562) 426-1842	Project:	Monarch Bay Community/Park Improvements, Monarch Bay Drive and PCH, Dana Point
		Figure 2	Local Geologic Map
		Proj. No.: 6925.20	Date: April, 2020

electrical substation, and streetlights. Other utilities, though not known at the time of this report preparation, may be present on site, and should be located and incorporated into site development plans accordingly.

3.0 FAULTING AND SEISMICITY

Dana Point, like the rest of southern California, is located within a seismically active region as a result of being located near the active margin between the North American and Pacific tectonic plates. The principal source of seismic activity is movement along the northwest-trending regional faults such as the San Andreas, San Jacinto, Newport-Inglewood and Whittier-Elsinore fault zones.

By the definition of CGS, an active fault is one which has had surface displacement within the Holocene Epoch (roughly the last 11,000 years). The CGS has defined a potentially active fault as any fault which has been active during the Quaternary Period (approximately the last 1,600,000 years). These definitions are used in delineating Earthquake Fault Zones as mandated by the Alquist-Priolo Geologic Hazard Zones Act of 1972 and as subsequently revised in 1997 as the Alquist-Priolo Earthquake Fault Zoning Act and Earthquake Fault Zones. The intent of the act is to require fault investigations on sites located within Special Studies Zones to preclude new construction of certain inhabited structures across the trace of active faults. The Site is not located within the Alquist-Priolo Earthquake Fault Zone.

Several sources were researched for information pertaining to site seismicity. The majority of data was obtained from the program, EQFAULT, by Blake (2000) that allows for an estimation of peak horizontal ground acceleration (PGA) using a data file of approximately 150 digitized California faults. This program compiles information including the dominant type of faulting within a particular region, the maximum earthquake magnitude each fault is capable of generating, the estimated slip-rate for each fault, and the approximate location of the fault trace. Printouts of the results of the fault search for the Site are shown as Plates I-1 and I-2 in Appendix B.

3.1 Deterministic Analysis

The Site is likely to be subject to strong seismic ground shaking during the life of the project. Based on the referenced literature and deterministic analysis performed with the EQFAULT software, the Newport-Inglewood (Offshore) Fault, approximately 2.1 miles (3.4 km) from the Site, would probably generate the most severe site ground motions. A Maximum Probable Earthquake (MPE), i.e. the maximum earthquake that is considered likely to occur during a 100-year time interval, of 7.1 Mw (moment magnitude as per USGS) has been assessed along the Newport-Inglewood (Offshore) Fault. As shown on Plate I-2 in Appendix B, estimated PGA resulting from an MPE event on the Newport-Inglewood (Offshore) Fault is on the order of 0.471g should this event occur at the fault's closest approach to the Site. Other nearby active faults include the San Joaquin Hills Fault and the Newport-Inglewood (L.A. Basin) Fault, located approximately 5.2

miles (8.3 km) and 14.0miles (22.5 km) away, respectively. In sum, approximately 34 active or potentially active faults have been found within 62 miles (100 km) of the Site.

3.2 Probabilistic Analysis

The seismicity of the Site was evaluated utilizing probabilistic analysis available from CGS (www.quake.ca.gov/gmaps/PSHA/psha_interpolator.html). The Maximum Probable Earthquake (MPE) and the Maximum Considered Earthquake (MCE) that carry 10 percent and 2 percent exceedance probabilities, respectively, in 50 years have been considered. Based on a typical damping ratio of 5% and a V_s^{30} value of 387 m/sec, derived from the "Set Site Parameters for Web Services"'' function as part of the "Hazard Spectrum Calculator (Local)" application available from the "OPENSHA" website, three spectral acceleration values representing peak ground acceleration (PGA), spectral acceleration for structural period of 0.2 second (Sa – 0.2 sec; typical of low-rise buildings) and spectral acceleration for structural period of 1.0 second (Sa – 1.0 sec; typical of multi-story buildings) have been analyzed and are tabulated below.

Seismic Acceleration Values from CGS's Ground Motion Interpolator (2008)

Latitude	Longitude	V_s^{30} (m/sec)	Scenario	Acceleration (g)		
				PGA	Sa – 1.0 sec	Sa – 1.0 sec
N 33.4869°	W 117.7280°	390	MPE [1]	0.400	0.751	0.342
			MCE [2]	0.683	1.531	0.757

1. MPE scenario carries a 10% exceedance probability in 50 years.
2. MCE scenario carries a 2% exceedance probability in 50 years.

3.3 2019 CBC Seismic Design Parameters

The earthquake design requirements listed in 2019 CBC and other governing standards account for faults classified as "active", in accordance with the most recent fault listing as per the United States Geological Survey (USGS) or the CGS. The seismic design of the proposed structures should be implemented in accordance with the applicable provisions stipulated in 2019 CBC unless otherwise specified by the governing authority having jurisdiction over the project. The 2019 CBC seismic design criteria for the Site based on a Site Class of "D", a Risk Category II and a scenario of Risk-Targeted Maximum Considered Earthquake (MCE_R) that carries a 2% exceedance probability in 50 years had been determined utilizing the OSHPD Seismic Design Maps web-application (http://seismicmaps.org) and the criteria stipulated in Chapters 11 and 12 of ASCE 7-16 (Reference 12). Summaries of the seismic coefficients for the Site are tabulated on the next page.

Please note that conformance to the 2019 CBC seismic design criteria does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not take place during the occurrence of a MCE_R event. The primary goal of seismic design is to protect life and not to avoid all damage, since such design may be economically prohibitive. Following a major earthquake, a building may be damaged beyond repair, yet not collapse. The Structural Consultant should review the pertinent parameters to evaluate the seismic design.

2019 CBC SEISMIC DESIGN PARAMETERS

Site Latitude:	N 33.4869°	Site Longitude:	W 117.7280°	Risk Category [a]	II
Seismic Parameter				**Recommended Value**	
Site Class [b]				D	
Soil Profile Name [b]				Stiff Soil	
Site Coefficient, F_a [c]				1.0	
Site Coefficient, F_v [d]				1.833	
0.2-Second Spectral Response Acceleration, S_S [e]				1.310g	
1.0-Second Spectral Response Acceleration, S_1 [f]				0.467g	
Adjusted 0.2-Second Spectral Response Acceleration, S_{MS} [g]				1.310g	
Adjusted 1.0-Second Spectral Response Acceleration, S_{M1} [h]				0.856g	
Design 0.2-Second Spectral Response Acceleration, S_{DS} [i]				0.874g	
Design 1.0-Second Spectral Response Acceleration, S_{D1} [j]				0.571g	
Long -Period Transition Period, T_L [k]				8 sec	
Mapped MCE_G Geometric Mean Peak Ground Acceleration, PGA [l]				0.576g	
Site Coefficient, F_{PGA} [m]				1.1	
MCE_G Peak Ground Acceleration adjusted for Site Class Effect, PGA_M [n]				0.633g	
Risk Category		**I or II or III**		**IV**	
Seismic Design Category based on SD_S [o]		D		D	
Seismic Design Category based on SD_1 [p]		D		D	

a Per 2019 CBC Table 1604.5
b Per 2019 CBC Section 1613.2.2
c Per 2019 CBC Table 1613.2.3(1). *Note: For Site Class "D", if simplified design procedure of Section 12.14 of ASCE 7-16 is adopted, the Fa value should be determined per Section 12.14.8.1 of ASCE 7-16 with no need for F_v, S_{MS}, S_{M1} values.*
d Per 2019 CBC Table 1613.2.3(2). *Note: For Site Class "D", the value is applicable provided C_s values are determined by Equations 12.8-2, 12.8-3 and 12.8-4 of ASCE 7-16.*
e Per 2019 CBC Figure 1613.2.1(1)
f Per 2019 CBC Figure 1613.2.1(2)
g Per 2019 CBC Equation 16-36
h Per 2019 CBC Equation 16-37
i Per 2019 CBC Equation 16-38
j Per 2019 CBC Equation 16-39
k Per ASCE 7-16 Figure 22-14
l Per ASCE 7-16 Figure 22-9
m Per ASCE 7-16 Table 11.8-1
n Per ASCE 7-16 Equation 11.8-1 = PGA x F_{PGA}
o Per 2019 CBC Table 1613.2.5(1)
p Per 2019 CBC Table 1613.2.5(2)

Please note, seismic design parameters for Site Classes "D", "E", and "F" should be obtained from site-specific seismic hazard analysis unless exceptions stipulated in Section 11.4.8 of ASCE 7-16 are invoked. The values listed in the table above reflect such exception invocation (see Footnotes c and d beneath the above table). If the structural design of the Improvements cannot be supporting by the invoked exceptions, the Geotechnical Consultant should be contacted for performing additional, site-specific seismic hazard analysis such that values of site-specific design parameters could be established.

4.0 GEOLOGIC HAZARDS

4.1 Surface Fault Rupture and Ground Shaking

No known active or potentially active faults are shown crossing the Site on published maps reviewed. No evidence for active faulting was observed on the Site during ASE's field investigation. The risk of surface rupture at the Site is considered very low. However, being in close proximity to several known active and potentially active faults, severe ground shaking should be expected during the life of the proposed development.

4.2 Seismic Hazards

4.2.1 Liquefaction:

As evidenced in Figure 3, Local Seismic Hazard Map, the Site and the surrounding area is not within an area identified as having a potential for soil liquefaction when subject to a MPE or MCE event.

The term "liquefaction" describes a phenomenon in which a saturated cohesionless soil loses strength and acquires a degree of mobility as a result of strong ground shaking during an earthquake. The factors known to influence liquefaction potential include soil type and depth, grain size, relative density, groundwater level, degree of saturation, and both the intensity and duration of ground shaking. The soils to the maximum explored depth of 14 feet 10 inches generally consist medium dense to dense granular soils and firm to stiff fine-grained soils.

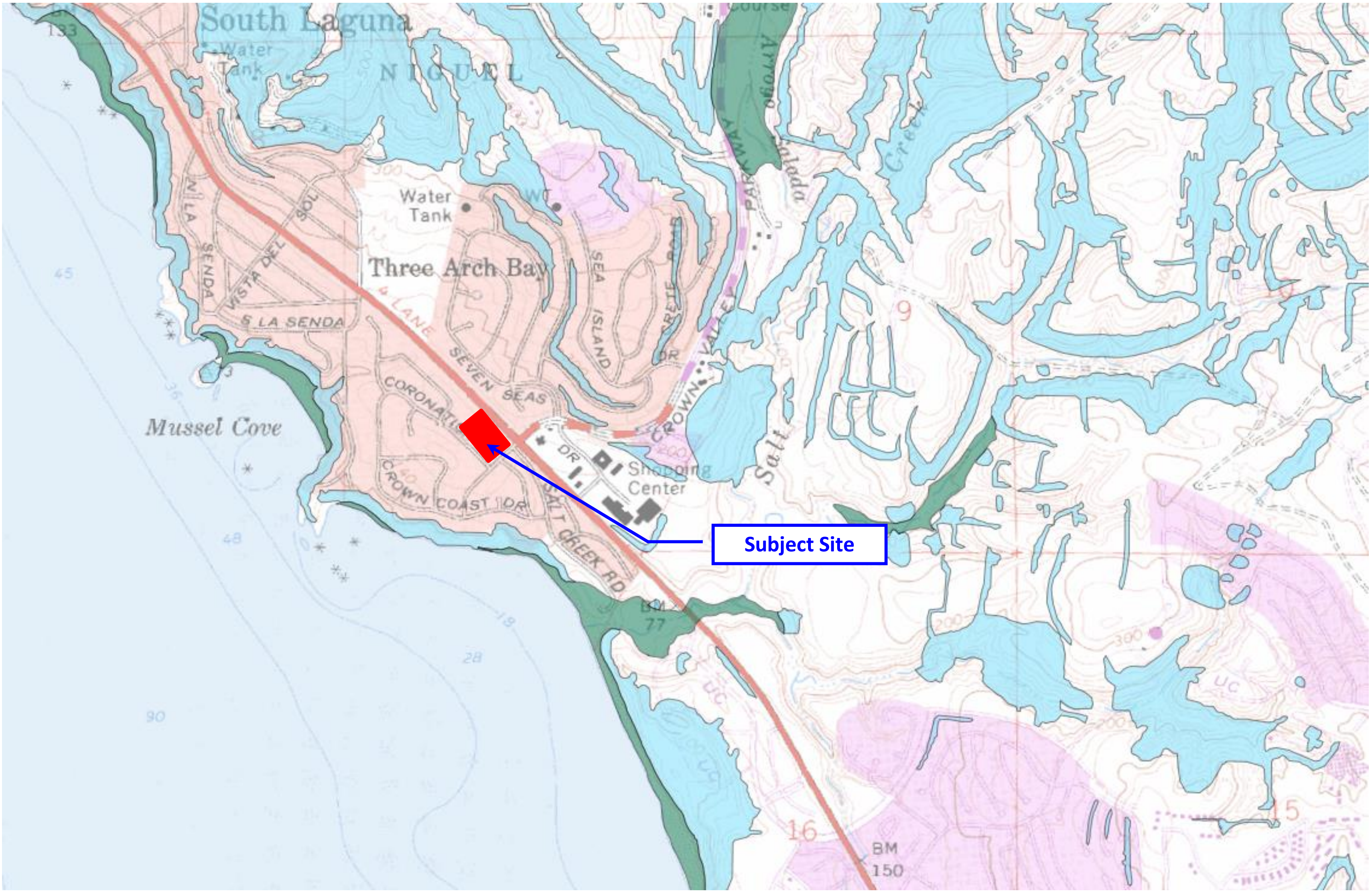
Considering that 1) groundwater was not encountered in Boring B-2 to a maximum explored depth of 14 feet 10 inches below existing grade, 2) historic high groundwater in a well nearest to the Site is greater than 68.61 feet below well grade (which is 30 feet higher than the Site grade), and 3) the existing site soils are predominantly in a medium dense to dense state within the depths explored, the likelihood of occurrence of seismically-induced liquefaction at the Site is deemed nil.

4.2.2 Earthquake-Induced Landslides:

There is no indication that recent landslides or unstable slope conditions exist on or adjacent to the Site that would otherwise result in an obvious landslide hazard to the Improvement construction or adjacent properties. ASE’s review of the same geohazard map that was based upon for the production of Figure 3 indicates that the Site is not located within an area identified as having a potential for earthquake-induced landslides. Due to lack of significant relief on or adjacent to the Site, the potential for earthquake-induced landslides at the Site is deemed nil.

4.2.3 Seismic Settlements:

Ground accelerations emitted from a seismic event can cause densification of loose soils both above and below the groundwater table that may result in settlements on ground surface due to volumetric compression of soil mass. This phenomenon is often referred to as seismic settlement and commonly takes place in relatively clean sands, as well as soils with low plasticity and less fines. As the site soils encountered in Borings B-1 through B-3 consist predominantly of medium dense to dense sands, these earth materials may undergo seismically-induced volumetric contraction above the historic high groundwater level (i.e. "dry" seismic settlement) upon impact of MPE/MCE events.



Partial Extract of Seismic Hazard Zones Official Map, Dana Point Quadrangle, California Geological Survey, 2001, dated December 21

LEGEND

- Approximate Site Location
- Potential Landslide Hazard Area
- Potential Liquefaction Hazard Area

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Project:	**Monarch Bay Community/Park Improvements, Monarch Bay Drive and PCH, Dana Point**
Figure 3	**Local Seismic Hazard Map**
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The "dry" seismic settlement on the Site, however, is not anticipated to exceed 1/2 inch. Such magnitude of "dry" seismic settlement is expected to affect relatively large area around the Site such that the differential settlement over short distance is likely to be small.

4.2.4 Lateral Spreading:

Lateral spreading, a phenomenon associating with seismically-induced soil liquefaction, is a display of lateral displacement of soils due to inertial motion and lack of lateral support during or post liquefaction. It is typically exemplified by the formation of vertical cracks on the surface of liquefied soils, and usually takes place on gently sloping ground or level ground with nearby free surface such as drainage or stream channel. Since the Site has been evaluated in Section 4.2.1 above not to be susceptible to seismically-induced liquefaction, the potential for the occurrence of liquefaction-induced lateral spreading is considered unlikely on the Site.

4.2.5 Tsunamis and Seiches:

As shown on Figure 4, Local Tsunami Hazard Map, excerpted from Reference 21, the Site <u>is not</u> in but is close to an area identified by CGS to be subject to potential tsunami inundation. Due to the elevation of the Site, hazard from tsunami is considered low.

Seiches are rhythmic movements of water within a lake or other enclosed or semi-enclosed body of water, generally caused by earthquakes. Since no lakes or other enclosed bodies of water lie on or near the Site, the hazard from seiches is not present at the Site.

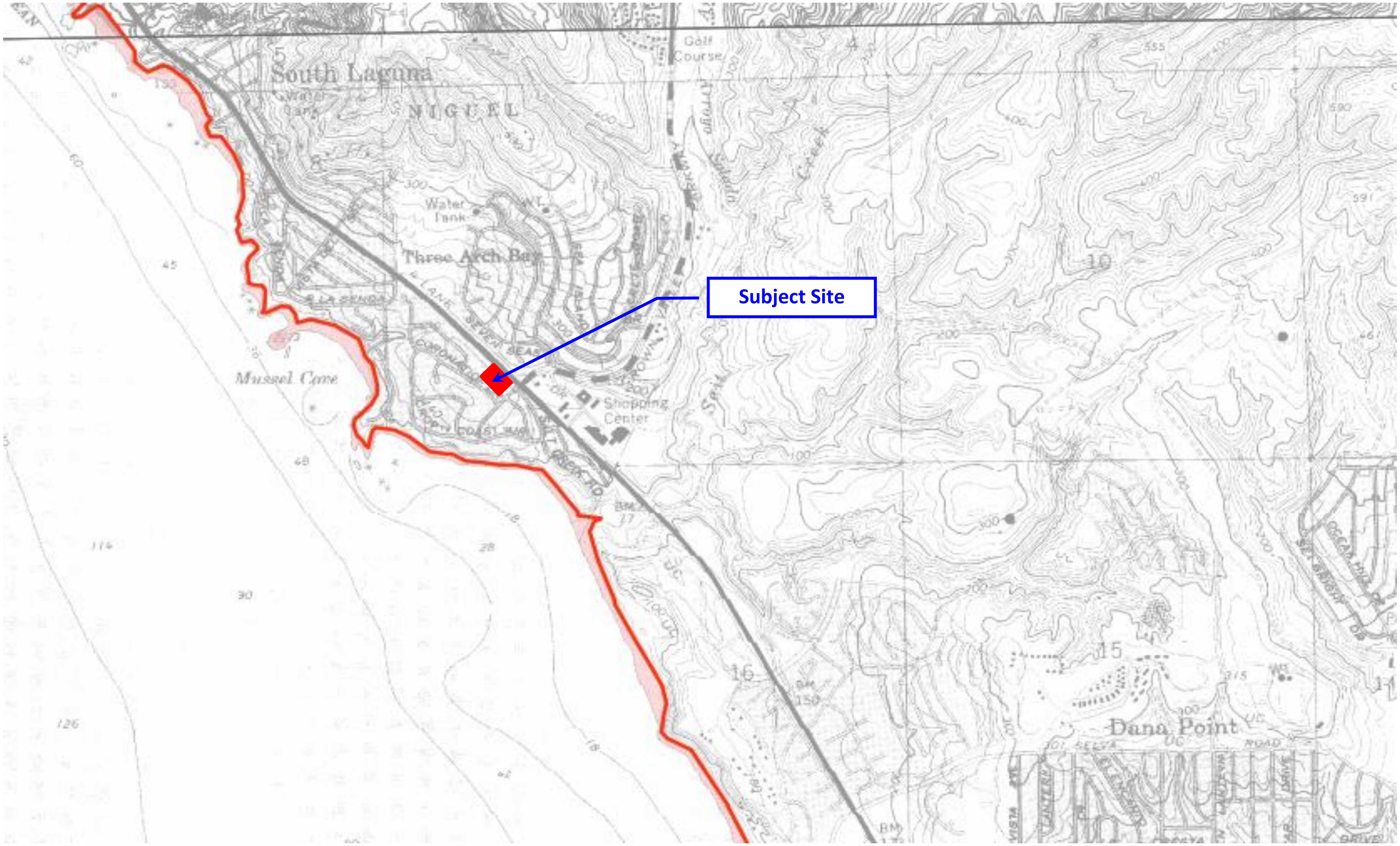
4.2.6 Flood Hazards:

Per FEMA Flood Insurance Rate Map (Map No. 06059C0501K, map revised March 21, 2019), the Site <u>is not</u> located within the 100-year floodplain (outside the area of 0.2% annual chance floodplain).

5.0 GEOTECHNICAL CONSIDERATIONS AND RECOMMENDATIONS

Based on ASE's field exploration, laboratory testing, engineering analysis, past successful experience and professional judgment, it is ASE's professional opinion that the major geotechnical considerations affecting the design and construction of the Improvements include the following:

1. Soil disturbances as a result of site demolition and clearing operations.
2. Presence of loose, low density soils within the zone of foundation bearing stratum.
3. Excavation and construction of new footings or flatworks located adjacent to or near existing right-of-way that might undermine stability.



(Partial Extract of the State of California Tsunami Innundation Map, Dana Point Quadrangle/San Juan Capistrano Quadrangle, dated March 15, 2009)

LEGEND

- Approximate Site Location
- Tsunami Innundation Line
- Tsunami Innundation Area

Associated Soils Engineering, Inc. **2860 Walnut Avenue** **Signal Hill, CA 90755** **Tel (562) 426-7990 Fax (562) 426-1842**	Project:	**Monarch Bay Community/Park Improvements, Monarch Bay Drive and PCH, Dana Point**	
	Figure 4	**Local Tsunami Hazard Map**	
	Proj. No.:	**6925.20**	Date: **April, 2020**

4. Presence of soils that exhibit "Medium" expansion potential at shallow depth near Boring B-5 that may heave or shrink noticeably and unevenly upon saturation and drying, respectively, resulting in potentially noticeable and uneven displacement of overlying foundations, structural improvements, flatworks and utilities.

In consideration of the above factors, it is ASE's opinion that overexcavation and backfilling with properly compacted fill in the footprint areas of the Improvements, as recommended herein, will be essential to reduce unfavorable foundation displacement as a consequence of settlement, moisture-induced volumetric contraction and/or heaves of underlying soils, and to provide satisfactory bearing stratum for supporting the foundations of the Improvements. The grading recommendations provided herein should be reviewed when final project concept and grading plans become available. It is assumed that the proposed finish grades will be close to existing site grades (± one foot).

Conventional shallow foundations comprising continuous spread footings and isolated pad footings, together with slab-on-grade, bearing on approved compacted fill soils are deemed suitable for structural support.

5.1 Site Preparation

5.1.1 Existing Improvements:

Prior to grading operations, it will be necessary to remove designated existing construction, including any remaining buried obstructions, which may be in the areas of Improvement construction. Structure removal should include foundations. Concrete flatwork and asphalt pavements, if present, should also be removed from the areas of proposed construction. Concrete and asphalt fragments from site demolition operations should be disposed of off-site.

5.1.2 Surface Vegetation:

Surface vegetation should be stripped from areas of proposed construction. Stripping should penetrate six (6) inches into surface soils. Any soil contaminated with organic matter (such as root systems or strippings mixed into the soil) should be disposed of off-site or set aside for future use in non-structural landscaped areas. Removal of trees and shrubs should include rootballs and attendant root systems.

5.1.3 Underground Utilities:

Any underground utilities to be abandoned within the zone of proposed construction should be cut off a minimum of five (5) feet from the area of the new structures. The ends of cut-off lines should be plugged a minimum of five (5) feet with concrete exhibiting minimum shrinkage characteristics to prevent water migration to or from hollow lines. Capping of lines may also be required should the plug be subject to any line pressure.

Alternatively, deep hollow lines may be left in place provided they are filled with concrete or 2-sack control density fill (slurry fill). No filled line should be permitted closer than two (2) feet from the bottom of future footings, unless it has been pre-approved by the Geotechnical Consultant. However, local ordinances relative to abandonment of underground utilities, if more restrictive, will supersede the above minimum requirements.

5.2 Site Grading

In view of reducing the adverse impact associated with the development of excessive total or differential settlement or heaves within subsurface soils underneath the Improvements, as well as to ensure uniform bearing competency for the foundations, preparation of on-site soils is recommended in the following sections.

5.2.1 Undocumented Fill/Disturbed Native Soils:

Although not observed in any of ASE's exploratory borings, any undocumented fill soil, if encountered during site grading in the building pad areas of the Improvements, as well as any native soils disturbed during demolition and clearing operations, should be excavated full depth under the observation and confirmation by the Geotechnical Consultant. Lateral extent of overexcavation beyond building or improvement perimeters, where possible, should be to a minimum distance equal to the depth of undocumented fill/disturbed soil encountered or three (3) feet, whichever is greater.

For other secondary improvements such as free-standing walls, hardscape or pavement, the lateral extent of removal, where possible, should be to a minimum distance equal to the depth of undocumented fill/disturbed soils encountered or eighteen (18) inches, whichever is greater.

The exposed excavation bottom should be scarified/reworked to a minimum nine (9) inches depth and recompacted to a minimum 90 percent relative compaction with a minimum moisture content of two (2) percentage points above optimum moisture content, prior to backfilling with approved soils as specified in Section 5.2.8. Unless otherwise stated, the measurement of relative compaction in this report should always refer to ASTM D1557-12 Test Method.

5.2.2 Expansive Soils:

Laboratory test results on near surface soil samples indicate "Very Low" and "Medium" soil expansion potential (i.e. Expansion Indices, EI = 4 and 55, respectively, per ASTM D4829-19 Test Method) as defined in 2019 CBC. Lightly loaded structural elements such as shallow foundations and slabs are likely to undergo noticeable movements, especially in areas underlain by site clayey soils with "Medium" expansion potential. Design provisions such as increased reinforcements, thicker slabs, deeper foundations, or other measures may help alleviate the effects of “Medium” soils expansion but may not completely eliminate the problem.

It is also desirable that the soil expansion potential be re-evaluated through additional testing during or after rough grading operations to verify the design adequacy of foundation and/or slab-on-grade against the re-tested soil expansion potential as heterogeneity within soil mass is not uncommon.

5.2.3 Remedial Grading:

Except for the proposed split face block wall that is anticipated to be located in area underlain by “Medium” expansive clayey soils, to provide acceptable support for structural foundations and slabs, it is recommended that on-site soils within the footprint of the Improvements be overexcavated and removed uniformly to a minimum depth of three (3) feet below existing grade, or one (1) foot below the bottom of the lowest footing, whichever is lower, and replaced with properly compacted fill such that the building/wall foundations and slabs are supported on a re-engineered, compacted fill layer. The excavation bottoms should be near uniform. The overexcavation should extend laterally to a minimum distance of three (3) feet beyond the perimeters of the Improvements, where possible.

The compacted fill within the Building pad areas should consist of “Very Low” expansive (EI ≤ 20), granular material, compacted to at least 90 percent relative compaction with minimum moisture content of two (2) percentage points above optimum moisture content. On-site subgrade soils in the park and entrance areas at their present state, generally exhibit a satisfactory EI and, thus, are deemed suitable for re-use as compacted fill.

The soils exposed at excavation bottoms to a depth of nine (9) inches should be scarified, reworked and recompacted to exhibit a minimum 90 percent relative compaction with a minimum moisture content of two (2) percentage points above the optimum moisture content, prior to receiving fill placement. The exposed excavation bottom should be observed, tested, and approved by the Geotechnical Consultant prior to placing compacted fill. In case of the presence of localized loose soils, the overexcavation needs to be deepened accordingly to delete the loose soil condition. However, this deepened overexcavation may be terminated when the exposed native, undisturbed soils exhibit a natural relative compaction greater than 85 percent, subject to the testing and inspection by the representative from the Geotechnical Consultant.

For the split face block wall that is to be located adjacent to Pacific Coast Highway, in view of the competence of the existing native soils, no additional overexcavation/recompaction beneath and beyond the footprint of the footing is deemed needed, provided that the exposed footing bottom is verified by the Geotechnical Consultant for its competency and the footing design criteria stipulated in Section 5.3.2 below are complied with.

The Geotechnical Consultant should be provided with appropriate foundation details and staking during grading to verify that depths and/or locations of the recommended overexcavation are adequate. For areas on site that grading recommendations stipulated in both Sections 5.2.1 and 5.2.3 apply, the more stringent grading criteria between the two sections should govern.

The depth of overexcavation should be reviewed by the Geotechnical Consultant during the actual construction. Any subsurface obstruction, buried structural elements, and unsuitable material encountered during grading, should be immediately brought to the attention of the Geotechnical Consultant for proper exposure, removal, and processing, as recommended.

5.2.4 Temporary Excavation:

Excavations of site soils four (4) feet or deeper should be temporarily shored or sloped in accordance with Cal OSHA requirements.

a) Temporary Sloping:

In areas where excavations deeper than four (4) feet are not adjacent to existing structures of public right-of-ways, sloping procedures may be utilized for temporary excavations. It is recommended that temporary slopes in native soils be graded no steeper than 1.5:1 (H:V) for excavations up to ten (10) feet in depth. The above temporary slope criteria is based on level soils conditions behind temporary slopes with no surcharge loading (structures, traffic) within a lateral distance behind the top of slope equivalent to the slope height.

It is recommended that excavated soils be placed a minimum lateral distance from top of slope equal to the height of slope. A minimum setback distance equivalent to the slope height should be maintained between the top of slope and heavy excavation/grading equipment.

Should running sand conditions be experienced during excavation operations, flattening of cut slope faces, or other special procedures may be required to achieve stable, temporary slopes. Soil conditions should be reviewed by the Geotechnical Consultant as excavation progresses to verify acceptability of temporary slopes. Final temporary cut slope design will be dependent upon the soil conditions encountered, construction procedures and schedule.

b) Temporary Shoring:

Temporary shoring will be required for those excavations where temporary sloping as specified above is not feasible.

Temporary cantilever shoring, if used, should be designed to resist active earth pressures of 37 and 59 pounds per cubic foot (pcf) equivalent fluid pressure (EFP) for site granular and clayey soils, respectively, for level soil conditions behind shoring. The resultant lateral deflection of shoring and surficial settlement immediately behind shoring are estimated to be on the order of one (1) to one and one half (1 ½) percent of the shored excavation depth. Should this ground deformation be intolerable to the existing structure, ASE should be consulted for more detailed analysis and further recommendations.

The design shoring should also include surcharge loading equivalent to one-third (1/3) of the loading of existing structures and anticipated traffic, including delivery and construction equipment, when loading is within a distance from the shoring equal to the depth of excavation. In addition, a minimum uniform lateral pressure of 100 pounds per square foot (psf) in the upper ten (10) feet of shoring should be considered in the design when normal traffic is permitted within ten (10) feet of the shoring.

5.2.5 Exterior Slab-on-Grade/Flatwork/Hardscape/Pavement Subgrade Preparation:

For the purpose of reducing future unsightly and uneven movements and cracks of any newly re-constructed exterior slab-on-grade, flatwork, hardscape, or pavement, it is recommended that the subgrade soils to eighteen (18) inches below the bottom of and eighteen (18) inches laterally beyond the footprint of exterior concrete slab-on-grade/flatwork/hardscape/pavement should be overexcavated and recompacted to at least 90 percent relative compaction with a minimum moisture content of two (2) percentage points above optimum moisture content. Prior to placement of the above recommended fill layer, the upper six (6) inches of exposed native subgrade should be reworked to at least 90 percent relative compaction with a minimum moisture of two (2) percentage points above optimum moisture content.

From geotechnical viewpoint, new landscape area with only softscape is not subject to subgrade preparation and remedial grading requirements mentioned in Sections 5.2.1, 5.2.3 and 5.2.5.

5.2.6 Suitable Soils and Imported Soils:

Unless otherwise stated, any soils re-used or imported as fill for the completion of grading operations required for the Buildings in the park and entrance should consist of predominantly “Very Low” expansive (EI ≤ 20), granular material, and should be exhibiting a relatively uniform gradation, free of debris, particles greater than 4 inches in maximum dimension, organic matter or other deleterious materials.

Unless otherwise approved by the Geotechnical Consultant, the fill materials should also comply with the following soil corrosivity criteria for the desired concrete and reinforcement protection.

Corrosivity Criteria for Select Fill and General Fill			
Soluble Sulfate (% by weight) (1)	**Soluble Chloride (ppm) (2)**	**Resistivity Value (ohm-cm) (3), (4)**	**pH-Value (4)**
≤ 0.1	≤ 100	≥ 2000	7.0 ~ 8.8

(1) California Test Method 417. (2) California Test Method 422. (3) ASTM G187-18 Test Method. (4) California Test Method 643.

Imported fill soils or base materials should be examined by a representative of this office and tested as necessary for evaluating their suitability for use as fill prior to being hauled to the Site. Final acceptance of any imported soil will be based upon review and testing of the soil actually delivered to the Site. All blended soils to be used as fill must be tested and approved by the Geotechnical Consultant prior to being used for fill placement. All soils and base material should be subjected to continuing verification testing during site grading.

5.2.7 Backfilling and Compaction Requirements:

Existing site soils at their present state and composition, unless indicated or tested otherwise, are considered suitable for re-use as fill during site grading, provided they are 1) free of debris, particles greater than 4 inches in maximum dimension, organic matter or other deleterious materials, 2) are not environmentally contaminated, 3) adequately moisture conditioned to permit achieving the required compaction, and 4) foundations and slabs are designed and constructed as per recommendations and criteria stipulated in Sections 5.3 and 5.4 below. No nesting of large particles (2- to 4-inch size) should be permitted during backfilling operations.

On-site soils and import materials approved for use as fill should be placed in horizontal lifts not exceeding 8 inches in loose thickness, moisture conditioned to a minimum of two (2) percentage points above optimum moisture content, and compacted to a minimum 90 percent relative compaction per ASTM D1557-12 Test Method, unless otherwise stated.

5.2.8 Tests and Observations:

All subgrade preparation, compaction, and backfill operations should be performed under the observation of and testing by the Geotechnical Consultant's field representative. An adequate number of field tests should be taken to ensure compliance with this report and local ordinances.

If it is determined during grading that site soils require overexcavation to greater depths for obtaining proper support for the proposed structures, this additional work should be performed in accordance with the recommendations of the Geotechnical Consultant.

5.3 Foundation Design

It is ASE's opinion that conventional continuous spread footings and isolated pad footings bearing on approved compacted fill soils may be used to provide foundation support for the Improvements, provided that the site grading recommendations presented in Section 5.2 above are incorporated in project planning and design, and implemented during site construction. Presented below are the recommended geotechnical design and construction criteria for conventional footing foundation and slab-on-grade.

5.3.1 Conventional Shallow Footing Foundation - New Buildings:

a) Minimum Footing Dimension and Reinforcement:

In order to mobilize sufficient soils bearing capacity supporting the new footings for the Buildings, it is recommended that the minimum footing embedments, widths and reinforcements for various footing types tabulated below be considered.

Minimum Footing Dimension & Reinforcement					
Continuous Spread Footing/Strip Footing			**Isolated Pad Footing**		
Depth (in) (1)	**Width (in)**	**Reinforcement (2)**	**Depth (in) (1)**	**Width (in)**	**Reinforcement (2)**
12	12	Two #4 bars – one near the top and one near the bottom	12	18 square	Two #4 bars – one near the top and one near the bottom, applied bi-axially

(1) Footing embedment measured from the nearest adjacent lowest soils grade
(2) Based strictly from geotechnical point of view.

Foundation design details such as concrete strength, reinforcements, etc. should be established by the Structural Consultant.

b) Allowable Soils Bearing Capacity:

For footings complying with the minimum dimension requirements stipulated in Section 5.3.1 a) above, the allowable soils bearing capacities, inclusive of both dead and live loads, should be as per tabulated below:

Allowable Soils Bearing Capacity (psf)		**Increase per 12-inch Increment in Footing Width (psf)**	**Increase per 12-inch Increment in Footing Depth (psf)**	**Maximum Composite Ceiling Value (psf)**
Continuous Spread Footing/ Strip Footing	**Isolated Pad Footing**			
2,000	2,000	150	500	3,000

The allowable bearing capacities tabulated above may be increased by one-third (1/3) when subject to short-term, transient loading induced by wind or seismic activities.

c) Lateral Resistance:

Resistance to lateral loads can be assumed to be provided by passive lateral earth pressure and by friction acting on structural components in permanent contact with the subgrade soils.

For site preparation implemented as per recommended in the above Section 5.2, lateral resistance on the sides of foundations may be computed using a passive lateral earth pressure of 230 pcf EFP for footings embedded into approved compacted fill soils, subject to a maximum of 2300 psf. An ultimate coefficient of friction on the order of 0.4 may also be used for structural dead load acting between the footing bottom and the supporting soils. The passive lateral earth pressure may be used in conjunction with the ultimate coefficient of friction in calculating composite lateral resistance, provided the passive lateral earth pressure value is reduced by one-third (1/3). The composite lateral resistance may be increased by one-third (1/3) under short term, transient wind or seismic loading.

d) Static Settlements:

Total static settlements resulting from compression of subgrade soils for conventional footings designed and constructed in accordance with the above criteria, and supporting maximum assumed dead plus live (D+L) column and wall loads mentioned in Section 1.1.2 above, are not anticipated to exceed three-quarter (3/4) inch, upon implementation of site preparation as per recommended in Section 5.2 above. A differential settlement on the order of one-quarter (1/4) inch over a distance of 30 feet is anticipated between similarly loaded adjacent isolated pad footings, as well as for continuous wall footings over a distance of approximately 30 feet.

Please be reminded that the Geotechnical Consultant should be contracted for further evaluation and recommendations, as necessary, should final design structural loads exceed the maximum loads assumed in the above analyses by more than ten (10) percent.

5.3.2 Conventional Shallow Footing Foundation - Split-Face Block Wall:

a) Minimum Footing Dimension and Reinforcement:

In order to mobilize sufficient soils bearing capacity supporting the new footings for the Improvements, it is recommended that the minimum footing embedments, widths and reinforcements for various footing types tabulated on the next page be considered.

Minimum Footing Dimension & Reinforcement					
Continuous Spread Footing/Strip Footing			Isolated Pad Footing		
Depth (in) (1)	Width (in)	Reinforcement (2)	Depth (in) (1)	Width (in)	Reinforcement (2)
36	15	Four #4 bars – two near the top and two near the bottom	36	24 square	Four #4 bars – two near the top and two near the bottom, applied bi-axially

(1) Footing embedment measured from the nearest adjacent lowest soils grade
(2) Based strictly from geotechnical point of view.

Foundation design details such as concrete strength, reinforcements, etc. should be established by the Structural Consultant.

b) Allowable Soils Bearing Capacity:

For footings complying with the minimum dimension requirements stipulated in Section 5.3.2 a) above, the allowable soils bearing capacities, inclusive of both dead and live loads, should be as per tabulated below:

Allowable Soils Bearing Capacity (psf)		Increase per 12-inch Increment in Footing Width (psf)	Increase per 12-inch Increment in Footing Depth (psf)	Maximum Composite Ceiling Value (psf)
Continuous Spread Footing/ Strip Footing	Isolated Pad Footing			
2,500	2,500	N/A	250	4,000

The allowable bearing capacities tabulated above may be increased by one-third (1/3) when subject to short-term, transient loading induced by wind or seismic activities.

c) Lateral Resistance:

Resistance to lateral loads can be assumed to be provided by passive lateral earth pressure and by friction acting on structural components in permanent contact with the subgrade soils.

For site preparation implemented as per recommended in the above Section 5.2, lateral resistance on the sides of foundations may be computed using a passive lateral earth pressure of 160 pcf EFP for footings embedded into approved compacted fill soils, subject to a maximum of 1600 psf. An ultimate coefficient of friction on the order of 0.2 may also be used for structural dead load acting between the footing bottom and the supporting soils. The passive lateral earth pressure may be used in conjunction with the ultimate coefficient of friction in calculating composite lateral resistance, provided the passive lateral earth pressure value is reduced by one-third (1/3). The composite lateral resistance may be increased by one-third (1/3) under short term, transient wind or seismic loading.

d) Static Settlements:

Total static settlements resulting from compression of subgrade soils for conventional footings designed and constructed in accordance with the above criteria, and supporting maximum assumed dead plus live (D+L) column and wall loads mentioned in Section 1.1.2 above, are not anticipated to exceed three-quarter (3/4) inch, upon implementation of site preparation as per recommended in Section 5.2 above. A differential settlement on the order of one-quarter (1/4) inch over a distance of 30 feet is anticipated between similarly loaded adjacent isolated pad footings, as well as for continuous wall footings over a distance of approximately 30 feet.

Please be reminded that the Geotechnical Consultant should be contracted for further evaluation and recommendations, as necessary, should final design structural loads exceed the maximum loads assumed in the above analyses by more than ten (10) percent.

5.3.3 Retaining Walls:

It is ASE's understanding that there is no retaining wall planned as part of the Improvements construction. If design or planning change requires the construction of retaining wall, ASE should be consulted for pertinent retaining wall design parameters and construction guidelines.

5.3.4 Footing/Foundation Observation:

All footing/foundation excavations should be observed by the Geotechnical Consultant's representative to verify minimum embedment depths and competency of bearing soils. Such observations should be made prior to placement of any reinforcing steel or concrete.

5.4 Slabs-On-Grade

Concrete slab-on-grade for the Improvements and exterior flatwork/hardscape should be supported on properly compacted soils as recommended in Section 5.2 above. The slab subgrade soils should also be proof-rolled just prior to construction to provide a firm, unyielding surface, especially if the subgrade has been disturbed or loosened by the passage of construction traffic. Final compaction and testing of slab subgrade should be performed just prior to placement of concrete.

For structural design of concrete slabs in areas of granular soils, a modulus of subgrade reaction ("k") on the order of 150 pounds per square inch per inch ("psi/in") and an allowable bearing capacity of 900 psf may be used. Interior and exterior slabs should be properly designed and reinforced for the construction and service loading conditions. To minimize slab distress, geotechnically, it would be prudent to provide a minimum actual slab thickness of four (4) inches with minimum reinforcement consisting of number 3 reinforcing bars spaced maximum 18 inches on centers each way placed at mid-slab, or equivalent. The structural details, such as slab thickness, concrete strength, amount and type of reinforcements, joint spacing, etc., should be established by the Structural Consultant in accordance with pertinent sections in 2019 CBC.

The entirety of any new slabs each Improvement structure should be underlain by an impermeable vapor barrier (minimum 10-mil-thick visqueen). A minimum 12-inch overlap between visqueen sheets should be ensured during placement. All visqueen sheets should be puncture free prior to slab construction, and should be sandwiched top and bottom by two (2) inches of clean sand (Sand Equivalent, SE, ≥ 30 per ASTM D2419-14 Test Method). The concrete slab shall consist of a concrete mix design which will address bleeding, shrinking and curling.

Exterior slabs should be properly jointed to limit the number of concrete shrinkage cracks. For long/thin sections, such as sidewalks, expansion or control joints should be provided at spacing intervals equal to the width of the section. Slabs between 5 and 10 feet in minimum dimension should have a control joint at centerline. Slabs greater than 10 feet in minimum dimension should have joints such that unjointed sections do not exceed 10 feet in maximum dimension. Where flatwork adjoins structures, it is recommended that a foam joint or similar expansion material be utilized. Joint depth and spacing should conform to the ACI recommendations. It is, however, cautioned that uneven heaving of exterior slabs may develop in the future when prolonged irrigation or seepage permeates the subgrade soil, especially in areas that expansive soil pockets exist due to inadequate control or inspection of earthwork construction.

5.5 Pool Design and Construction

a) Due to the “Very Low” expansion potential of site soils at the potential location of the pool, pool and/or spa walls should be designed to resist an equivalent fluid pressure of 60 pounds per cubic foot (pcf). The bottom of the pool should be supported by a thickened slab based on an allowable bearing capacity of 900 pounds per square foot (psf), and a modulus of subgrade reaction (“k-value”) of 150 pcf.

b) The soil exposed at the bottom of the pool excavation should be cleaned of any debris, scarified to a depth of twelve (12) inches, and recompacted to a minimum 90 percent of the maximum dry density at two (2) percentage points above optimum moisture content per ASTM D1557-12 Test Method, unless otherwise stated.

c) Any portion of the pool within fifteen (15) feet from the top of a descending slope should be designed to be capable of supporting water without soil support.

d) Hydrostatic relief valves should be incorporated into the pool and spa design.

e) All fittings and pipe joints, particularly those in the side of the pool or spa, should be properly sealed to prevent water from leaking into the underlying soils.

f) An elastic waterproof expansion joint should be installed to prevent water from seeping into the soil at all deck joints.

g) It is the opinion of ASE that a properly constructed pool will not require any subdrainage. The Geotechnical Consultant should be on-site during the pool construction to inspect conditions, to evaluate the excavation and, if necessary, to provide any recommendations.

5.6 Interlocking Concrete Pavers

ASE understands the reconfigured entry area is to be furnished with non-permeable interlocking concrete pavers. Site subgrade soils in the areas of paver installation should be prepared as outlined in Section 5.2.5 of the Soils Report. The following section has been developed using assumptions based on the Structural Design of Interlocking Concrete Pavement for Municipal Streets and Roadways (Reference 23) and ICPI manual "Permeable Interlocking Concrete Pavements", 3rd Edition (Reference 22), and a conservative R-value of 40 (equivalent to a CBR of 8).

Location	"R"-Value / CBR	Paver Thickness (in.)	Bedding Sand* Thickness (in.)	AASHTO No. 57 Aggregate Base (in.)	AASHTO No. 2 Stone Subbase (in.)
All Traffic Surfaces	40/8	3.15	1.0	4.0	8.0

*Bedding Sand should have a minimum tested Sand Equivalent (S.E.) value of 30 per ASTM 2419-14 Test Method.

Th upper twelve (12) inches of subgrade materials should be compacted to a minimum 90 percent relative compaction at a minimum one (1) percentage point above optimum moisture content per ASTM D1557-12 Test Method.

If the paver areas are to be used during construction, or if the type and frequency of traffic is greater than assumed in the design, the paver structural section should be re-evaluated for the anticipated traffic.

5.7 Asphaltic Concrete (AC) Flexural Pavement Design

The finish grade at the subject site is anticipated to be underlain by compacted structural fill consisting of site soils. With a test R-value of 65 shown in Appendix A, for preliminary pavement design purposes, a conservative R-Value of 40 has been utilized considering the potential heterogeneity within site silty sand soils. Three (3) traffic indices ("TI") of 4.5, 5.5 and 7.0, together with the conservative R-Value, have been utilized for the development of preliminary recommendations for the pavement sections. Analyses performed in accordance with the current edition of the Caltrans Highway Design Manual, and assuming compliance with site preparation recommendations, it is recommended that the AC pavement structural sections tabulated on the next page be considered:

Traffic Index (TI)	Pavement Section Alternatives		Remark
	AC (inches)	**AB** [(1)] (inches)	
4.5	3.0	4.0	For auto parking stalls.
5.5	3.0	5.5	For auto circulation aisles.
7.0	3.5	8.0	For fire lanes and truck access ways/entry and exits.
	4.0	7.0	

(1) CAB or CMB, per Green Book sections 200-2.2 and 200-2.4, respectively, compacted to at least 95% relative compaction.

Please be reminded that the preliminary pavement section recommendations have been established based purely on procedures stipulated in Caltrans Manual. Local government authority should be consulted for minimum pavement section requirements and, if more stringent than that recommended by ASE, be complied with.

It is recommended that R-Value testing be performed on representative soil samples after rough grading operations on the upper two (2) feet to confirm/modify applicability of the above pavement sections.

The aggregate base should conform to the criteria of Crushed Aggregate Base (CAB) or CMB stipulated in Sections 200-2.2 or 200-2.4 of the Greenbook, respectively. The base course should be compacted to a minimum relative compaction of 95% at a minimum of one (1) percentage point above the optimum moisture content. Field testing should be used to verify compaction, aggregate gradation, and compacted thickness.

The asphalt concrete pavement should be compacted to 95% of the unit weight as tested in accordance with the Hveem procedure. The asphalt concrete material shall conform to Type III, Class C2 or C3, of the Greenbook. All subgrade and aggregate base materials should be proof-rolled by heavy rubber tire equipment to verify that the subgrade and base grade are in a non-yielding condition.

If the paved areas are to be used during construction, or if the type and frequency of traffic is greater than assumed in the design, the pavement section should be re-evaluated for the anticipated traffic.

5.8 Portland Cement Concrete (PCC) Pavements

The concrete pavement sections are based on load safety factors of 1.0 and 1.1, and a modulus of subgrade reaction ("k" Value) of 150 pounds per cubic inch for site soils compacted as subgrade material, and the design procedures presented in the Portland Cement Association bulletin "Thickness Design for Concrete Highway and Street Pavements" (EB109.01P), 1984. A design service life of 20 years was assumed for the design of the Portland cement concrete pavement section.

Concrete Flexural Strength (psi) (1)	Pavement Thickness (in) (2), (4)	Pavement Thickness (in) (3), (4)
600	5.0	6.0
650	4.5	5.5

(1) Represents 90-day flexural strength. Based on Figure 10 of Reference 15, concrete with 28-day unconfined compressive strength values of 4000 to 4500 psi typically correlates to 90-day flexural strength values of 600 and 650 psi, respectively.
(2) Load Safety Factor = 1.0 (Auto Parking Stalls)
(3) Load Safety Factor = 1.1 (Fire Lanes/Truck Traffic Areas/Entry and Exits)
(4) Assumes no PCC shoulder or curb.

The Structural Consultant should establish the design details of the concrete pavement section, including reinforcements, concrete strength, and joint and load transfer requirements.

It is recommended that edges of concrete pavements which are not adjacent to existing buildings, or are adjacent to planter areas, be downturned a minimum of 12 inches or be constructed with curbing to prevent water infiltration to subgrade soils. If edges are downturned or curbing is constructed, the above pavement thicknesses should be decreased by 1/2 inch.

The upper one (1) foot of exposed subgrade soils beneath concrete pavements should be further compacted to a minimum 95 percent relative compaction with a minimum moisture content of two (2) percentages point above optimum moisture content. Subgrade soils should exhibit a firm, unyielding surface in addition to the recommended compaction. Final compaction and testing of pavement subgrade should be performed just prior to placement of aggregate base and/or concreting. Other pertinent subgrade preparation measures stipulated in the "Thickness Design for Concrete Highway and Street Pavements" (EB109.01P), 1984, or required by the jurisdictional municipal authorities should be followed accordingly.

5.9 Site Drainage

Per Section 1804.4 of 2019 CBC, a minimum 5% descending gradient away from the Improvements for a minimum distance of 10 feet should be incorporated for earth grade placed adjacent to the foundation. This descending gradient may be reduced to 2% for any impervious areas, such as concrete paved walkways, within the 10-foot zone. For areas where the 10-foot drainage distance is not attainable, alternative measure such as concrete-lined swales having a minimum 2% gradient may be adopted to divert the water away from the Improvements, provided that a minimum 5% gradient is maintained in the distance between the building footprint and the diversion measure such as swales. For more specific site drainage guidelines, the Project Civil Consultant should refer to the pertinent sections in 2019 CBC.

Any planter areas to be placed adjacent to structure perimeters should be provided with impervious bottoms and a drainage pipe, or should be planted with drought tolerant plants, to divert water away from foundation and slab subgrade soils. Excessive moisture variations in site soils could result in significant volume changes and movement.

5.10 Soil Corrosivity Evaluation

Soils corrosivity tests were performed on a representative sample of site soil. These tests are meant to determine the corrosive potential of on-site soils to proposed concrete foundations/flatwork and underground metal conduit. The soils corrosivity test results are presented in Appendix A.

5.10.1 Concrete Corrosion:

Disintegration of concrete may be attributed to the chemical reaction of soils sulfates and hydrated lime and calcium aluminate with the cement. The severity of the reaction resulting in expansion and disruption of the cement is primarily a function of the concentration of soluble sulfates and the water-cement ratio of the concrete.

Soluble sulfate contents of 0.033% and 0.002% by weight have been recorded from testing per California Test Method (CTM) 417 conducted on on-site soils, as indicated in Appendix A. As per Table 19.3.1.1 of ACI 318-19, soils exhibiting soluble content less than 0.1% by weight are classified as having "Not Applicable" sulfate exposure and "S0" sulfate exposure category. As such, for structural features to be in direct contact with on-site soils, the requirements regarding the type of Portland cement or water-cement ratio pertinent to the tested "S0" sulfate exposure category, as per stipulated in Table 19.3.1.1 of ACI 318-19, should be followed.

5.10.2 Metal Corrosion:

In the evaluation of soil corrosivity to metal, the hydrogen ion concentrates (pH) and the electrical resistivity of the site and backfill soils are the principal variables in determining the service life of ferrous metal conduit. The pH of soil and water is a measure of acidity or alkalinity, while the resistivity is a measure of the soils resistance to the flow of electrical current.

Currently available design charts indicate that corrosion rates decrease with increasing resistivities and increasing alkalinities. It can also be noted that for alkaline soils, the corrosion rate is more influenced by resistivity than by pH.

The resistivity values of 5,971 and 640 ohm-cm per ASTM G187-18 Test Method coupled with respective pH-values of 8.61 and 8.76, per CTM 643 classifies the on-site soils tested to be mildly corrosive and very corrosive, respectively, to buried ferrous metals. Based on CTM 643, the year to perforation for 18-gauge steel in contact with soils of similar resistivity and pH-value is greater than <u>50</u> years and <u>21</u> years for the mildly corrosive and very corrosive on-site soils, respectively. In lieu of additional testing, alternative piping materials, i.e. plastic piping, may be used instead of metal if longer service life is desired or required for utility pipes and fittings in direct contact with on-site soils, especially in area near Boring B-5. These resistivity values of on-site soils may also have implications to other building materials and depths of embedment for steel reinforcement, etc. It is recommended that a qualified corrosion consultant be engaged to review the building plans.

A soluble chloride content 17 ppm was recorded in our laboratory tests per CTM 422 for two (2) separate samples. Per Caltrans guidelines and specifications (References 18 and 19), soils exhibiting soluble chloride contents exceeding 500 ppm are considered "corrosive". The soils are thus classified as "non-corrosive" per Caltrans criterion, and the special measure in terms of rebar protection against chloride corrosion under Exposure Class "C0" stipulated in Tables 19.3.1.1 and 19.3.2.1 of ACI 318-19 should be complied with.

5.11 Utility Trenches

All trenches should be backfilled with approved fill material compacted to relative compaction of not less than 90 percent of maximum dry density (ASTM D1557-12 Test Method). Care should be taken during backfilling to prevent utility line damage.

The on-site soils may be used for backfilling utility trenches from one (1) foot above the top of pipe to the surface, provided the material is free of organic matter and deleterious substances. Any soft and/or loose materials or fill encountered at pipe invert should be removed and replaced with properly compacted fill or adequate bedding material.

Some on-site soils may be suitable for bedding or shading of utilities, subject to further SE testing per ASTM D2419-14 Test Method. Site or imported soils for pipe bedding should consist of non-expansive granular soils having a tested SE value not less than 30.

If sandy soils are used for trench backfill, the backfill should be topped with a minimum 2-foot thick cap of compacted fine-grained soil. Also, a minimum 10-foot length of trench at the entrance and exit points of the Improvements should be backfilled with fine-grained soils to serve as a plug to prevent water migration into structure foundation support zones.

The walls of temporary construction trenches may not to be stable when excavated nearly vertical, due to the potential for caving. Shoring of excavation walls or flattening of slopes will be required if excavation depths greater than 4 feet are necessary. Please note that trenches should be located so as not to impair the bearing capacity of soils or cause settlement under foundations. As a guide, trenches parallel to foundations should be clear of a 45-degree plane extending outward and downward from the edge of the foundations. All work associated with trenches, excavations and shoring must conform to the latest Cal OSHA requirements.

5.12 Plan Review, Observations and Testing

When foundation and grading plans are completed, they should be forwarded to the Geotechnical Consultant for review of conformance with the intent of these recommendations.

All excavations should be observed by a representative of this office to verify minimum embedment depths, competency of bearing soils and that the excavations are free of loose and disturbed materials. Such observations should be made prior to placement of any fill, reinforcing steel or concrete. All grading and fill compaction should be performed under the observation of and testing by a Geotechnical Consultant or his representative.

6.0 CLOSURE

This report has been prepared for the exclusive use of **Monarch Bay Association** (the Client) and their subconsultants for use in design and construction of the Improvements at the Site. The report has not been prepared for use by other parties and may not contain sufficient information for purposes of other parties.

The Client is responsible for ensuring the information and recommendations contained in this report are brought to the attention of the Owner or the other design consultants, incorporated into the project plans, and implemented by project contractors. This report should be named on project plans as a part of the project specifications.

We request and recommend notification should any of the following occur:

1. Final plans for site development indicate utilization of areas not originally proposed for construction.
2. Structural loading conditions vary from those utilized for evaluation and preparation of this report.
3. The site is not developed within 12 months following the date of this report.

If changes or delays do occur, this office should be notified and provided with finalized plans of site development for our review to enable us to provide the necessary recommendations for additional work and/or updating of the report. Any charges for such review and necessary recommendations would be at the prevailing rate at the time of performing review work.

The findings contained in this report are based upon our evaluation and interpretation of the information obtained from the limited number of test borings and the results of laboratory testing and engineering analysis. As part of the engineering analysis it has been assumed, and is expected, that the geotechnical conditions existing across the area of study are similar to those encountered in the test excavations. However, no warranty is expressed or implied as to the conditions at locations or depths other than those excavated. Should conditions encountered during construction differ significantly from those described in this report, this office should be contacted immediately for recommendations prior to continuation of work.

Our findings and recommendations were obtained in accordance with generally accepted current professional principles and local practice in geotechnical engineering and reflect our best professional judgment. We make no other warranty, either express or implied.

These recommendations are, however, dependent on the aforementioned assumption of uniformity and upon proper quality control of engineered fill and foundations. Geotechnical observations and testing should be provided on a continuous basis during grading at the site to confirm preliminary design assumptions and to verify conformance with the intent of our recommendations. If parties other than Associated Soils Engineering, Inc. are engaged to provide geotechnical services during construction, they must be informed that they will be required to assume complete responsibility for the geotechnical phase of the project by either concurring with the recommendations in this report or providing alternative recommendations.

This concludes our scope of services as indicated in our proposal dated February 17, 2020, however, our report is subject to review by the controlling authorities for the project. Any further geotechnical services that may be required of our office to respond to questions/comments of the controlling authorities after their review of the report will be performed on a time-and-expense basis as per our current fee schedule. We would not proceed with any response to report review comments/questions without authorization from your office.

We appreciate your business and are prepared to assist you with construction-related services.

APPENDIX A

The following Appendix contains the substantiating data and laboratory test results to complement the engineering evaluations and recommendations contained in the report.

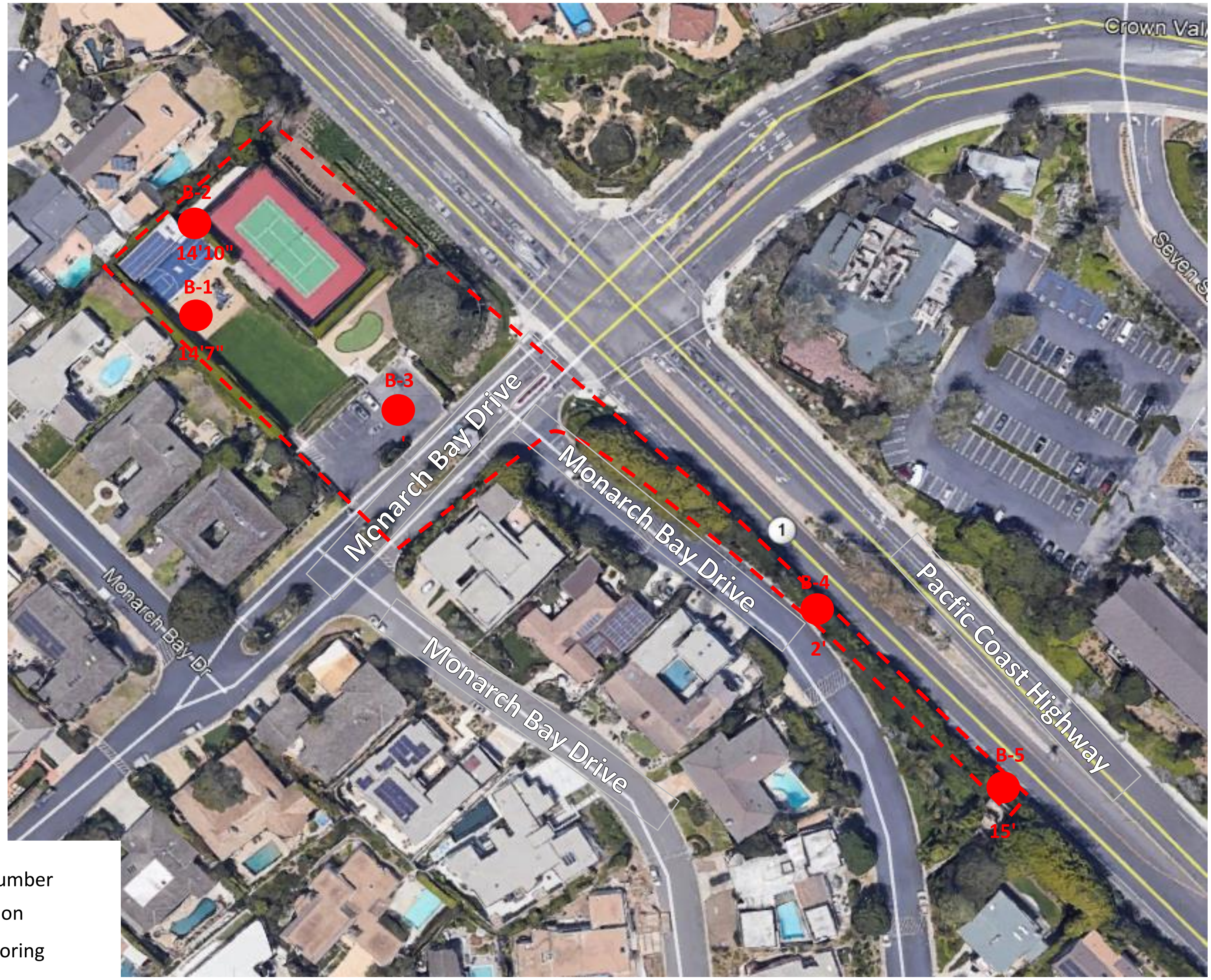
Site Exploration

On March 9, 2020, field explorations were performed by drilling five (5) test at the approximate locations indicated on the attached Boring Location Plan, Plate A. The exploratory borings were drilled and sampled by Associated Soils Engineering, Inc. (ASE), utilizing manually operated drilling/sampling equipment with 3-inch diameter cutting bucket bits. The borings extended to depths of 2 feet and 14 feet 10 inches below respective existing grade were reached in the exploratory borings.

Continuous observations of the materials encountered in the borings were recorded in the field. The soils were classified in the field by visual and textural examination and these classifications were supplemented by obtaining bulk soil samples for future examination in the laboratory. Relatively undisturbed samples of soils were extracted in 2.5-inch I.D. thin-walled Shelby tubes. All samples were secured in moisture-resistant bags immediately after retrieval from exploratory boring to minimize the loss of field moisture, followed by timely transportation to ASE's laboratory for ensuing testing. Upon completion of exploration, the borings were backfilled with excavated materials and compacted by tamping, with existing AC pavement patched with cold-patch asphalt.

Description of the soils encountered, depth of samples, field density and moisture content of tested samples, as well as respective laboratory tests performed are presented in the attached Field Logs of Borings ("B" Plates).

Plate A	Boring Location Plan
Plates B-1 through B-5	Field Logs of Borings



LEGEND

- B-1 Designated boring number
- ● Approx. boring location
- 14'7" Termoinal depth of boring
- Approx. site limit

Associated Soils Engineering, Inc. **2860 Walnut Avenue** **Signal Hill, CA 90755** **Tel (562) 426-7990 Fax (562) 426-1842**	**Boring Location Plan**	Project Name:	**Monarch Bay Community/Park Improvements, Monarch Bay Drive and PCH, Dana Point**
		Plate A	Proj. No.: **6925.20** Date: **April, 2020**

ASSOCIATED SOILS ENGINEERING, INC. Consulting Geotechnical Engineers

ASSOCIATED SOILS ENGINEERING, INC.

FIELD LOG OF BORING B-1

Sheet 1 of 1

Project: ***Proposed Park Improvements "Monarch Bay",Dana Point***

Location: ***Monarch Bay Drive & PCH*** | Project No. ***6925.20***

Dates(s) Drilled:	**3/9/20**	Logged By:	**Grant Zike**
Drilled By:	**Associated Soils Engineering,Inc.**	Total Depth:	**14 feet 7 inches**
Rig Make/Model:	**N/A**	Hammer Type:	**N/A**
Drilling Method:	**Hand Auger**	Hammer Weight/Drop:	**N/A**
Hole Diameter:	**3 inches**	Surface Elevation:	**N/A**

Comments: Groundwater not encountered.Backfill not determined

DEPTH (Ft.)	ELEVATION (MSL)	SAMPLE INTERVALS: BULK	SAMPLE INTERVALS: DRIVE	SAMPLE INTERVALS: TYPE, "N" or (Blows/ft.)	LITHOLOGY	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (Pcf)	MOISTURE CONTENT (%)	WELL COMPLETION	OTHER TESTS
0	0						WOOD CHIPS: 8.0"				
						SP-SM	SAND WITH SILT: Yellowish brown,damp,fine-grained sand	113.0	5.3		CONSOL,SHEAR
						SP-SM	same as above	105.6	8.1		
5	5					SP-SM	same as above,olive yellow	110.0	4.7		SHEAR
						SP	SAND: Pale yellow,dry,fine-grained sand	104.2	2.0		CONSOL
10	10										
						SP	same as above	104.2	2.0		

FIELD LOG OF BORING B-2

ASSOCIATED SOILS ENGINEERING, INC.

Project:	***Proposed Park Improvements "Monarch Bay",Dana Point***
Location:	***Monarch Bay Drive & PCH***
Project No.	***6925.20***

Dates(s) Drilled:	**3/9/20**	Logged By:	**Grant Zike**
Drilled By:	**Associated Soils Engineering,Inc.**	Total Depth:	**14 feet 10 inches**
Rig Make/Model:	**N/A**	Hammer Type:	**N/A**
Drilling Method:	**Hand Auger**	Hammer Weight/Drop:	**N/A**
Hole Diameter:	**3 inches**	Surface Elevation:	**N/A**

Comments: Groundwater not encountered.Backfill not determined

DEPTH (Ft.)	ELEVATION (MSL)	SAMPLE INTERVALS: BULK	SAMPLE INTERVALS: DRIVE	TYPE, "N" or (Blows/ft.)	LITHOLOGY	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (Pcf)	MOISTURE CONTENT (%)	WELL COMPLETION	OTHER TESTS
0	0					SP-SM	TOPSOIL: 3.0"				
							SAND WITH SILT: Yellowish brown,damp,fine-grained sand	107.4	4.9		MAX DENSITY, EXPANSION, REMOLD SHEAR, CORROSIVITY TESTS
						SP-SM	same as above	104.5	5.3		CONSOL
							same as above,olive yellow with yellow brown				
5	5					SP-SM	#no density possible	#	4.3		
						SP	SAND: Olive yellow,damp,fine-grained sand	105.9	3.6		
						SP	same as above,pale yellow	105.4	3.1		
10	10										
						SP	same as above	108.5	2.6		

ASSOCIATED SOILS ENGINEERING, INC.

FIELD LOG OF BORING B-3

Sheet 1 of 1

Project:	***Proposed Park Improvements "Monarch Bay", Dana Point***
Location:	***Monarch Bay Drive & PCH***
Project No.	***6925.20***

Dates(s) Drilled:	**3/9/20**	Logged By:	**Grant Zike**
Drilled By:	**Associated Soils Engineering,Inc.**	Total Depth:	**5 feet 4 inches**
Rig Make/Model:	**N/A**	Hammer Type:	**N/A**
Drilling Method:	**Hand Auger**	Hammer Weight/Drop:	**N/A**
Hole Diameter:	**3 inches**	Surface Elevation:	**N/A**

Comments: Groundwater not encountered. Backfill not determined

DEPTH (Ft.)	ELEVATION (MSL)	SAMPLE INTERVALS: BULK	SAMPLE INTERVALS: DRIVE	TYPE, "N" or (Blows/ft.)	LITHOLOGY	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (Pcf)	MOISTURE CONTENT (%)	WELL COMPLETION	OTHER TESTS
0	0						ASPHALTIC CONCRETE PAVEMENT: 6.0" to 3 lifts, 2 1/8" rubber at seam				
						SP-SM	SAND WITH SILT: Dark yellowish brown, damp, fine-grained sand				R-VALUE
5	5										

ASSOCIATED SOILS ENGINEERING, INC.

FIELD LOG OF BORING B-4

Project:	***Proposed Park Improvements "Monarch Bay",Dana Point***
Location:	***Monarch Bay Drive & PCH***
Project No.	***6925.20***

Dates(s) Drilled:	**3/9/20**	Logged By:	**Grant Zike**
Drilled By:	**Associated Soils Engineering,Inc.**	Total Depth:	**2 feet**
Rig Make/Model:	**N/A**	Hammer Type:	**N/A**
Drilling Method:	**Hand Auger**	Hammer Weight/Drop:	**N/A**
Hole Diameter:	**3 inches**	Surface Elevation:	**N/A**

Comments: Groundwater not encountered.Backfill not determined

DEPTH (Ft.)	ELEVATION (MSL)	SAMPLE INTERVALS BULK	SAMPLE INTERVALS DRIVE	TYPE, "N" or (Blows/ft.)	LITHOLOGY	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (Pcf)	MOISTURE CONTENT (%)	WELL COMPLETION	OTHER TESTS
0	0					SM	SILTY SAND: Dark brown,moist,fine-grained sand,with organics and tree roots				
			■					106.6	10.5		
							DRILLING NOTE:Tree roots encountered at 2 feet depth. Unable to auger further. Boring terminated.				

FIELD LOG OF BORING B-5

Sheet 1 of 1

ASSOCIATED SOILS ENGINEERING, INC.

Project: ***Proposed Park Improvements "Monarch Bay", Dana Point***

Location: ***Monarch Bay Drive & PCH*** | Project No. ***6925.20***

Dates(s) Drilled:	**3/9/20**	Logged By:	**Grant Zike**
Drilled By:	**Associated Soils Engineering, Inc.**	Total Depth:	**7 feet 2 inches**
Rig Make/Model:	**N/A**	Hammer Type:	**N/A**
Drilling Method:	**Hand Auger**	Hammer Weight/Drop:	**N/A**
Hole Diameter:	**3 inches**	Surface Elevation:	**N/A**

Comments: Groundwater not encountered. Backfill not determined

DEPTH (Ft.)	ELEVATION (MSL)	SAMPLE INTERVALS: BULK	SAMPLE INTERVALS: DRIVE	TYPE, "N" or (Blows/ft.)	LITHOLOGY	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (Pcf)	MOISTURE CONTENT (%)	WELL COMPLETION	OTHER TESTS
0	0						TOPSOIL: 8.0"				
						CL	SILTY CLAY: Very dark grayish brown, moist	115.4	14.5		MAX DENSITY, EXPANSION, REMOLD SHEAR, CORROSIVITY TESTS CONSOL@1'
						CL	SILTY CLAY WITH SAND: Brown and olive gray, moist, fine-grained sand, lens Clayey Sand (SC)	121.8	11.1		SHEAR
5	5					SC	CLAYEY SAND: Gray, moist, fine to medium-grained sand	126.8	11.1		CONSOL
							DRILLING NOTE: Rock(?) encountered at 7 feet 2 inches depth. Unable to auger further. Boring terminated.				

Laboratory Tests

After samples were visually classified in the laboratory, a testing program that would provide sufficient data for our evaluation was established.

- Moisture Content and Density Tests

The undisturbed soil retained within the Shelby tubes was tested in the laboratory to determine in-place dry density and moisture content. Test results are presented on the Field Logs of Borings (see attached "B" Plates).

- Consolidation and Direct Shear Tests

Consolidation (ASTM D2435-11) and direct shear (ASTM D3080-11) tests were performed on selected relatively undisturbed and remolded samples to determine the settlement characteristics and shear strength parameters of various soil samples, respectively. The results of these tests are shown graphically on the appended "C" and "D" Plates.

- Soil Corrosivity Tests

Tests of soluble sulfate and chloride contents were performed in accordance with the latest edition of California Test Methods 417 and 422, respectively, to assess the degree of corrosivity of the subgrade soils with regard to concrete and normal grade steel. Resistivity and pH-value tests were performed in accordance with the latest edition of ASTM G187-18 Test Method and California Test Method 643, respectively, to assess the degree of corrosivity of the subgrade soils with regard to ferrous metal piping. The test results are presented below.

Sample ID	Sulfate Content (1) (%)/ Degree of Severity	Chloride Content (2) (ppm) / Degree of Severity	Resistivity (3) (OHM-cm)/ Degree of Corrosivity	Ph-Value(4)
B-2 @ 0.25'-5'	0.033/Not Applicable	17/Not Applicable	5,971/Mildly Corrosive	8.61
B-5 @ 0.67'-5'	0.002/Not Applicable	17/Not Applicable	640/Very Corrosive	8.76

(1) California Test Method 417. (2) California Test Method 422. (3) ASTM G187-18 Test Method. (4) California Test Method 643.

- Maximum Dry Density/Optimum Moisture Content Tests

Maximum density tests were conducted in accordance with ASTM D1557-12 Test Method, Method A, using 5 equal layers, 25 blows each layer, 10-pound hammer, 18 inch drop in a 1/30 cubic foot mold. The results are as follows:

Sample ID	Maximum Dry Density (pcf)	Optimum Moisture Content (%)	Material Classification
B-2 @ 0.25'-5'	119.5	11.5	SM
B-5 @ 0.67'-5'	126.5	10.0	CL

Laboratory Tests – continued

- Expansion Tests

Expansion tests were performed on soil samples to determine the swell characteristics. The expansion tests were conducted in accordance with ASTM D4829-19 test procedures. The expansion samples were remolded to approximately 90 percent relative compaction at near optimum moisture content subjected to 144 pounds per square foot surcharge load and were saturated.

Sample ID	Molded Dry Density (pcf)	Molded Moisture Content (%)	% Saturation	Expansion Index (EI)	Expansion Classification
B-2 @ 0.25’-5’	111.5	11.2	58.8	4	Very Low
B-5 @ 0.67’-5’	115.3	10.4	60.9	55	Medium

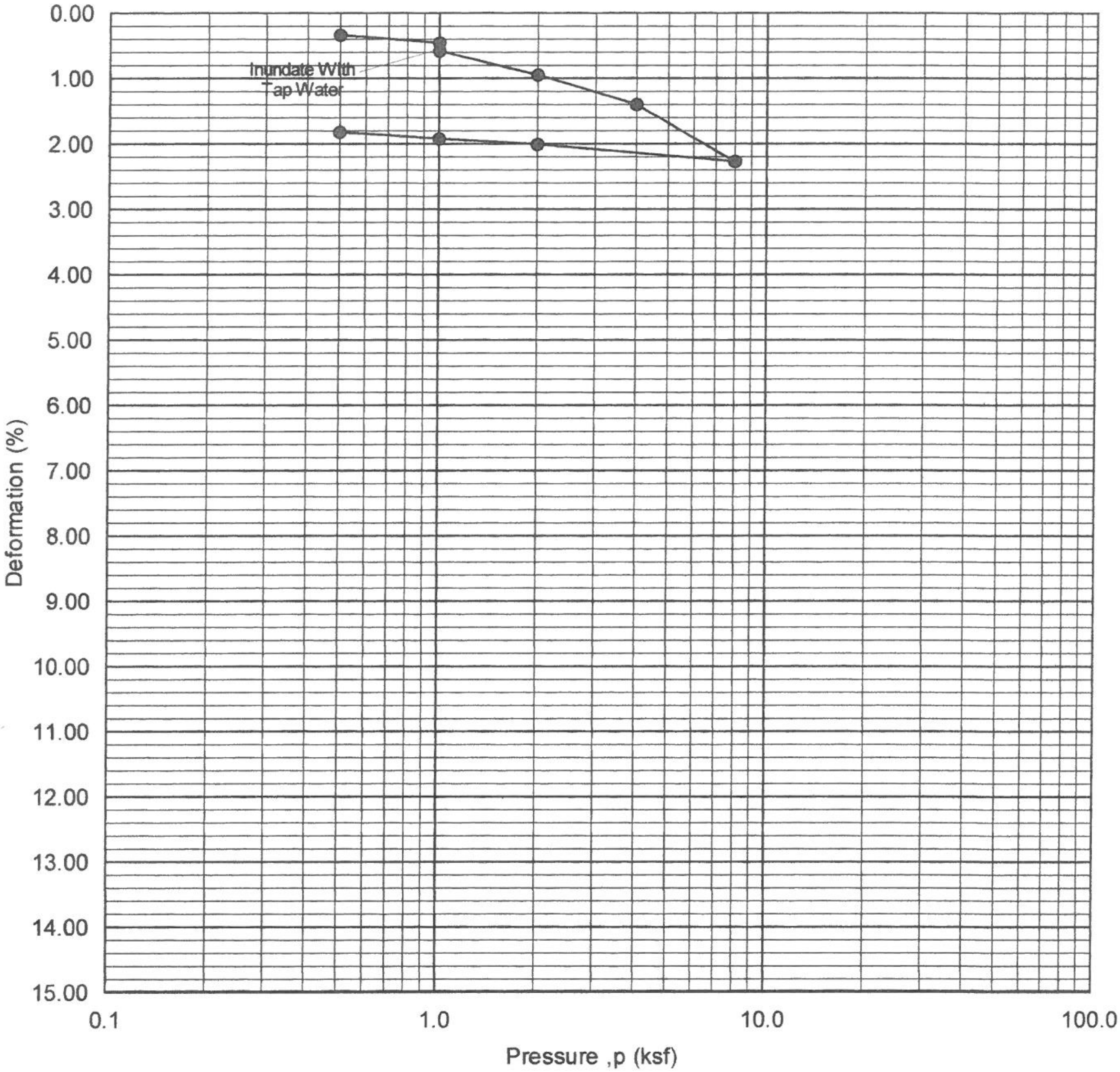
- "R" Value Analysis

The following "R" Value Stabilometer results were obtained in accordance with California 301 test procedures.

Stabilometer Results	Trial #1	Trial #2	Trial #3
Dry Density as molded, pcf	123.5	122.2	120.9
Moisture content as molded, %	7.7	8.2	9.0
Expansion Pressure, dial reading 10^4	7	1	0
Exudation Pressure, psi	685	405	150
Stabilometer "R" Value	83	68	61
Classification: Yellowish Brown Silty Fine Sand			
Source: Boring B-3 @ 0.5’-5'			
"R" Value equilibrium (300 psi Exudation Pressure) = 65			

Plates C-1 through C-5 Uni-axial Consolidation Test Results

Plates D-1 through D-5 Direct Shear Test Results

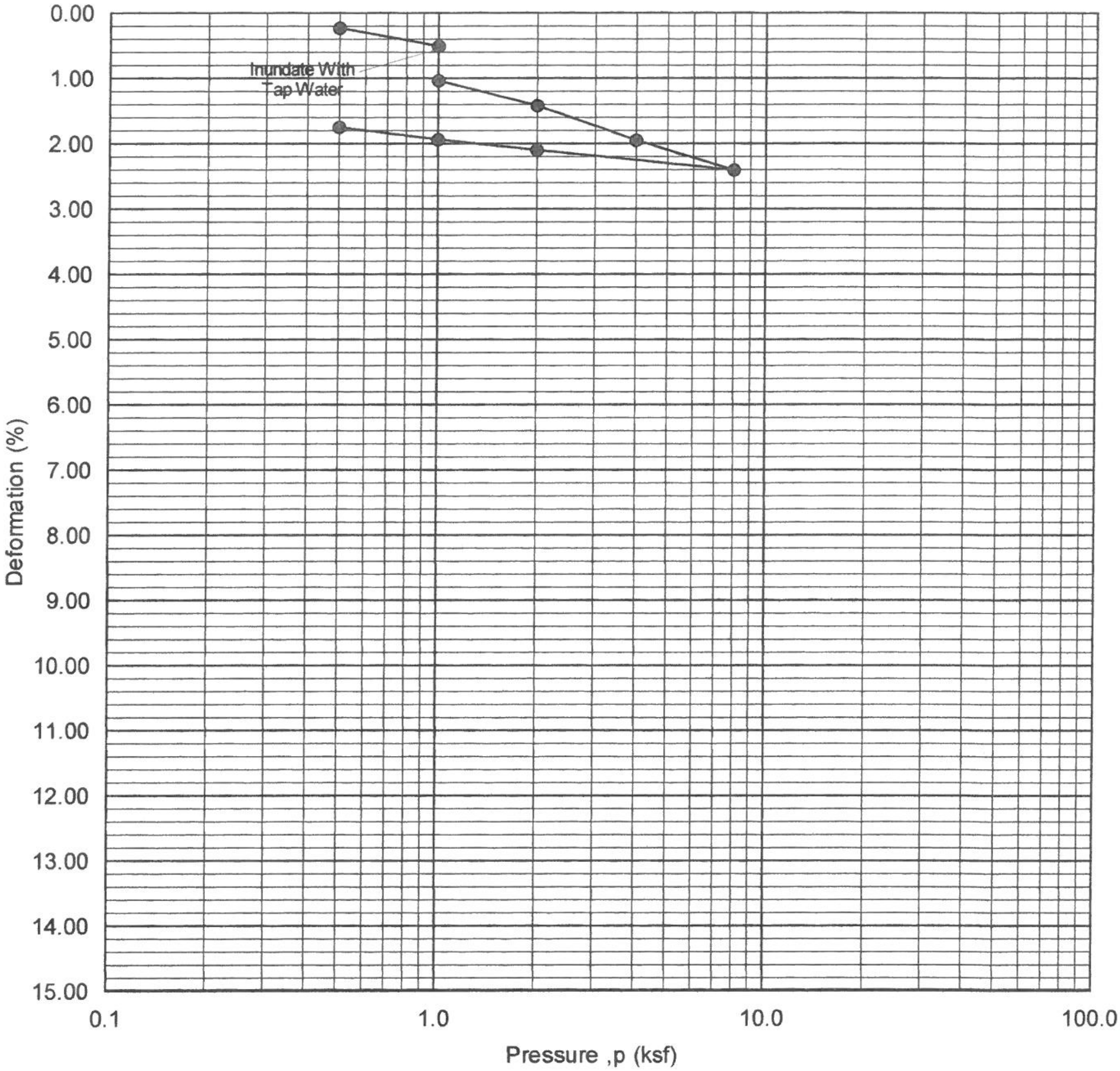


Boring No. : B-1
Depth (ft.) : 1.0
Sample Type: Fine Sand with Silt

Dry Density (pcf) = 113.0
Moisture (%) = 5.3

Project Name:Monarch Bay Entrance/Park Improvements,Dana Point	Project No.: 6925.20
ASSOCIATED SOILS ENGINEERING, INC.	ONE-DIMENSIONAL CONSOLIDATION PROPERTIES OF SOILS (ASTM D 2435)

PLATE C-1

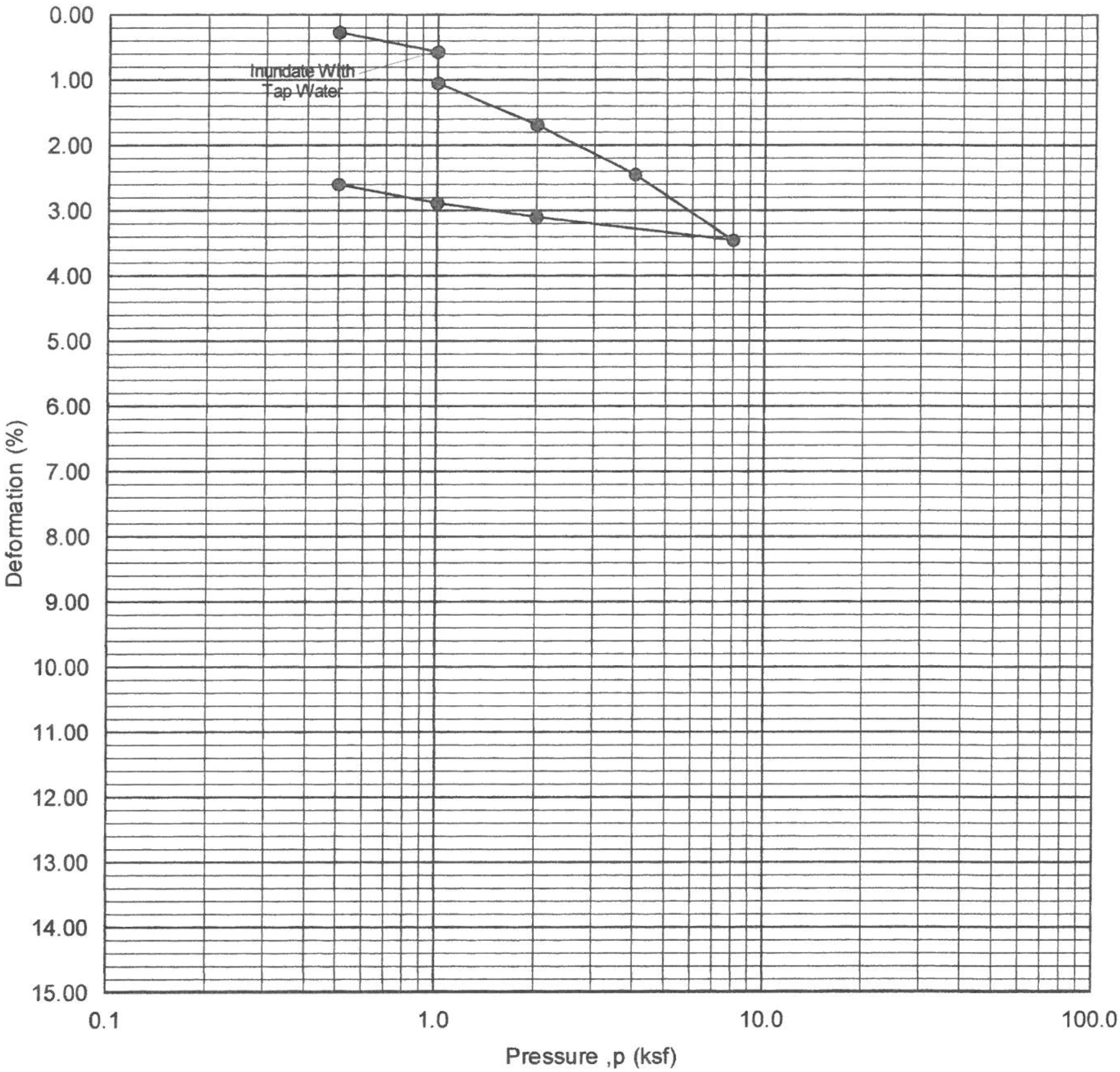


Boring No. : B-1
Depth (ft.) : 9.0
Sample Type: Fine Sand

Dry Density (pcf) = 104.2
Moisture (%) = 2.0

Project Name:Monarch Bay Entrance/Park Improvements,Dana Point	Project No.: 6925.20
ASSOCIATED SOILS ENGINEERING, INC.	ONE-DIMENSIONAL CONSOLIDATION PROPERTIES OF SOILS (ASTM D 2435)

PLATE C-2

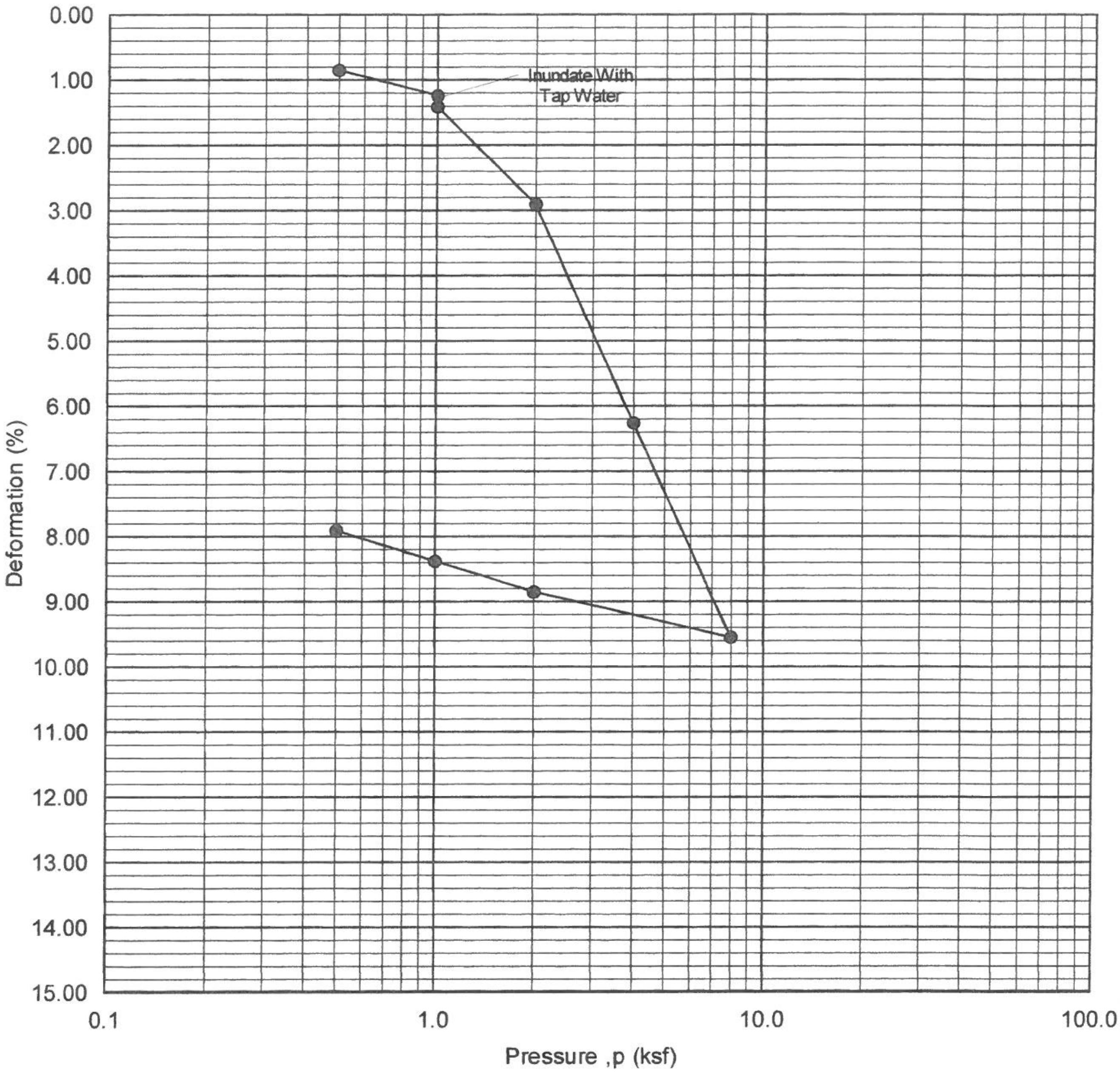


Boring No. : B-2
Depth (ft.) : 3.0
Sample Type: Fine Sand with Silt

Dry Density (pcf) = 104.5
Moisture (%) = 5.3

Project Name:Monarch Bay Entrance/Park Improvements,Dana Point	Project No.: 6925.20
ASSOCIATED SOILS ENGINEERING, INC.	ONE-DIMENSIONAL CONSOLIDATION PROPERTIES OF SOILS (ASTM D 2435)

PLATE C-3

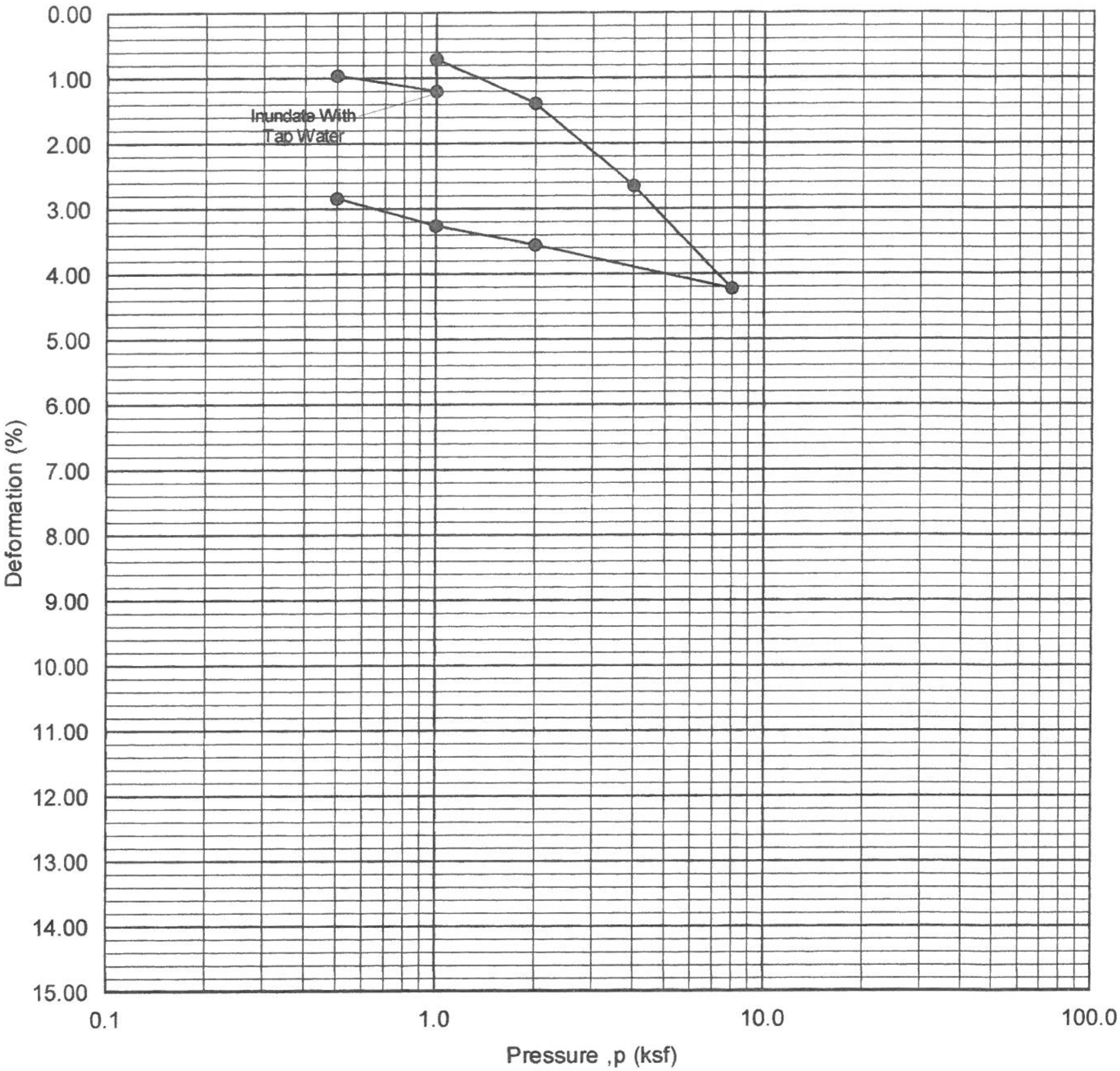


Boring No. : B-5
Depth (ft.) : 1.0
Sample Type: Silty Clay

Dry Density (pcf) = 115.4
Moisture (%) = 10.5

Project Name:Monarch Bay Entrance/Park Improvements,Dana Point	Project No.: 6925.20
ASSOCIATED SOILS ENGINEERING, INC.	ONE-DIMENSIONAL CONSOLIDATION PROPERTIES OF SOILS (ASTM D 2435)

PLATE C-4

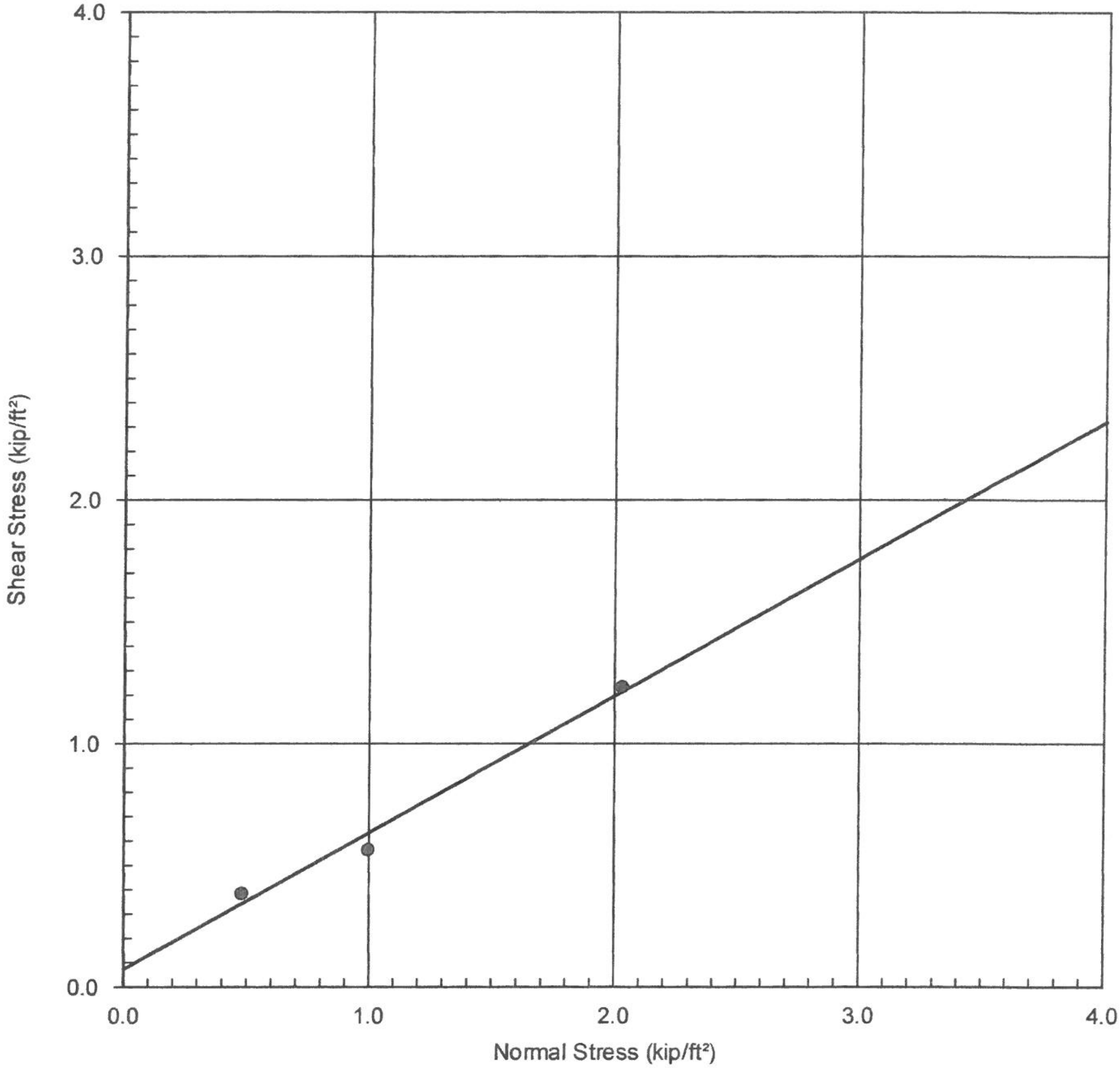


Boring No. : B-5
Depth (ft.) : 5.0
Sample Type: Clayey Fine to Medium Sand

Dry Density (pcf) = 126.8
Moisture (%) = 11.1

Project Name:Monarch Bay Entrance/Park Improvements,Dana Point	Project No.: 6925.20
ASSOCIATED SOILS ENGINEERING, INC.	ONE-DIMENSIONAL CONSOLIDATION PROPERTIES OF SOILS (ASTM D 2435)

PLATE C-5

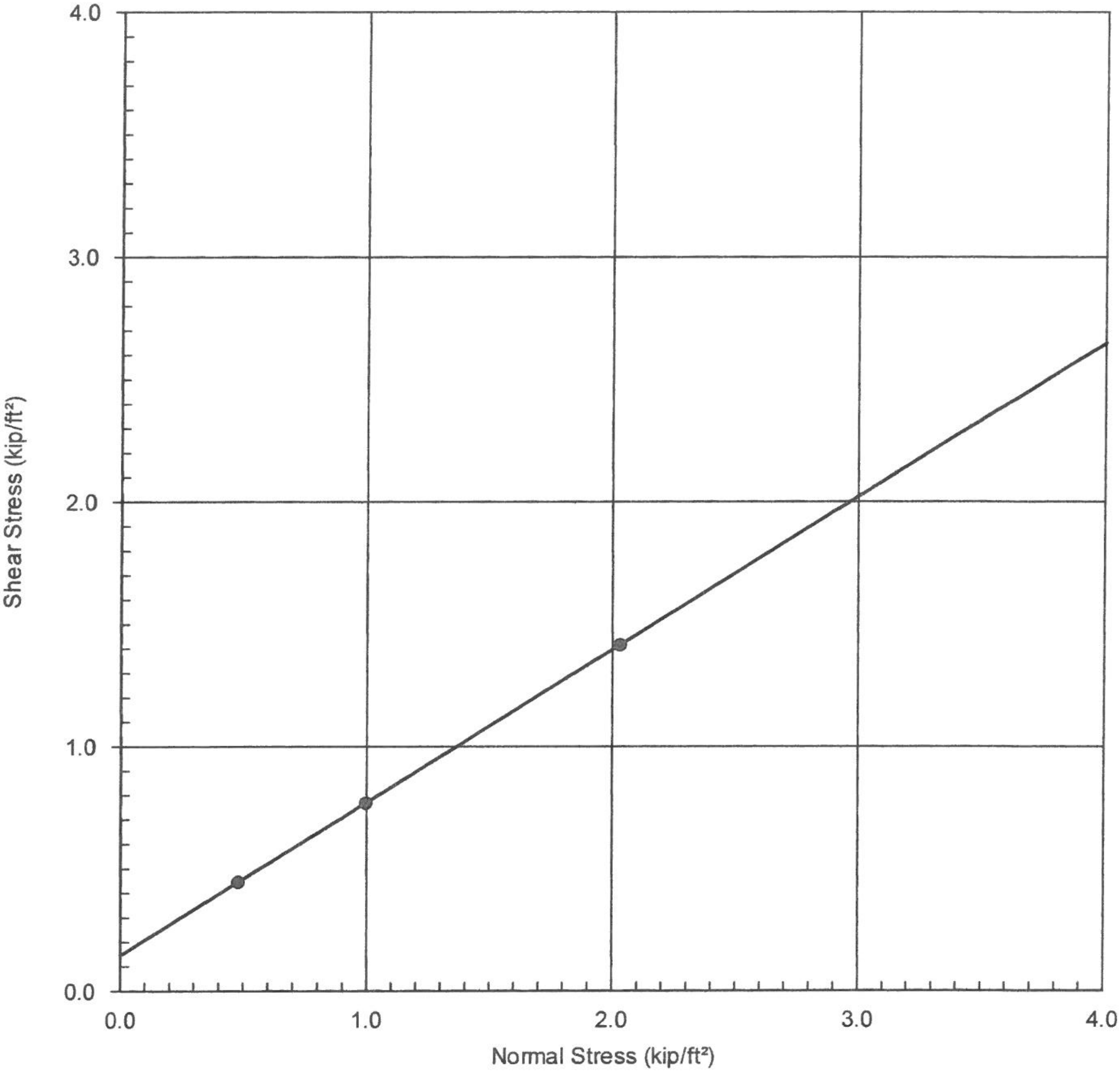


Boring No. : B-1
Depth (ft.) : 1.0
Sample Type : Relatively Undisturbed
Soil Type : Fine Sand with Silt

Cohesion(C) = 20 psf
Friction (ϕ) = 30°
Dry Density (pcf) = 113.0
Moisture (%) = 5.3

Project Name: Monarch Bay Entrance/Park Improvements, Dana Point	Project No.: 6925.20
ASSOCIATED SOILS ENGINEERING, INC.	DIRECT SHEAR TEST RESULTS (ASTM D 3080)

PLATE D-1

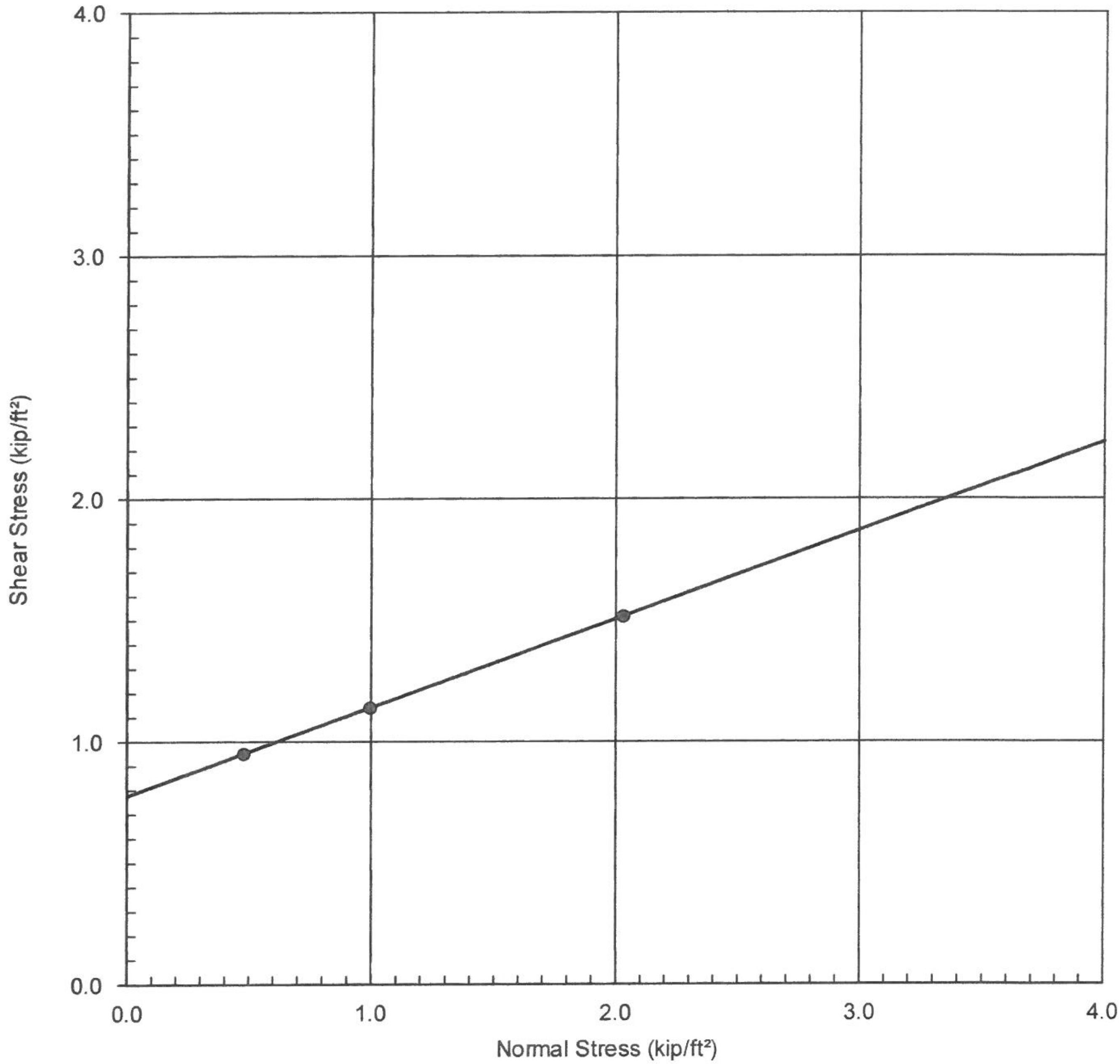


Boring No. : B-1
Depth (ft.) : 5.0
Sample Type : Relatively Undisturbed
Soil Type : Fine Sand with Silt

Cohesion(C) = 145 psf
Friction (ϕ) = 32°
Dry Density (pcf) = 110.0
Moisture (%) = 4.7

Project Name: Monarch Bay Entrance/Park Improvements, Dana Point	Project No.: 6925.20
ASSOCIATED SOILS ENGINEERING, INC.	DIRECT SHEAR TEST RESULTS (ASTM D 3080)

PLATE D-2

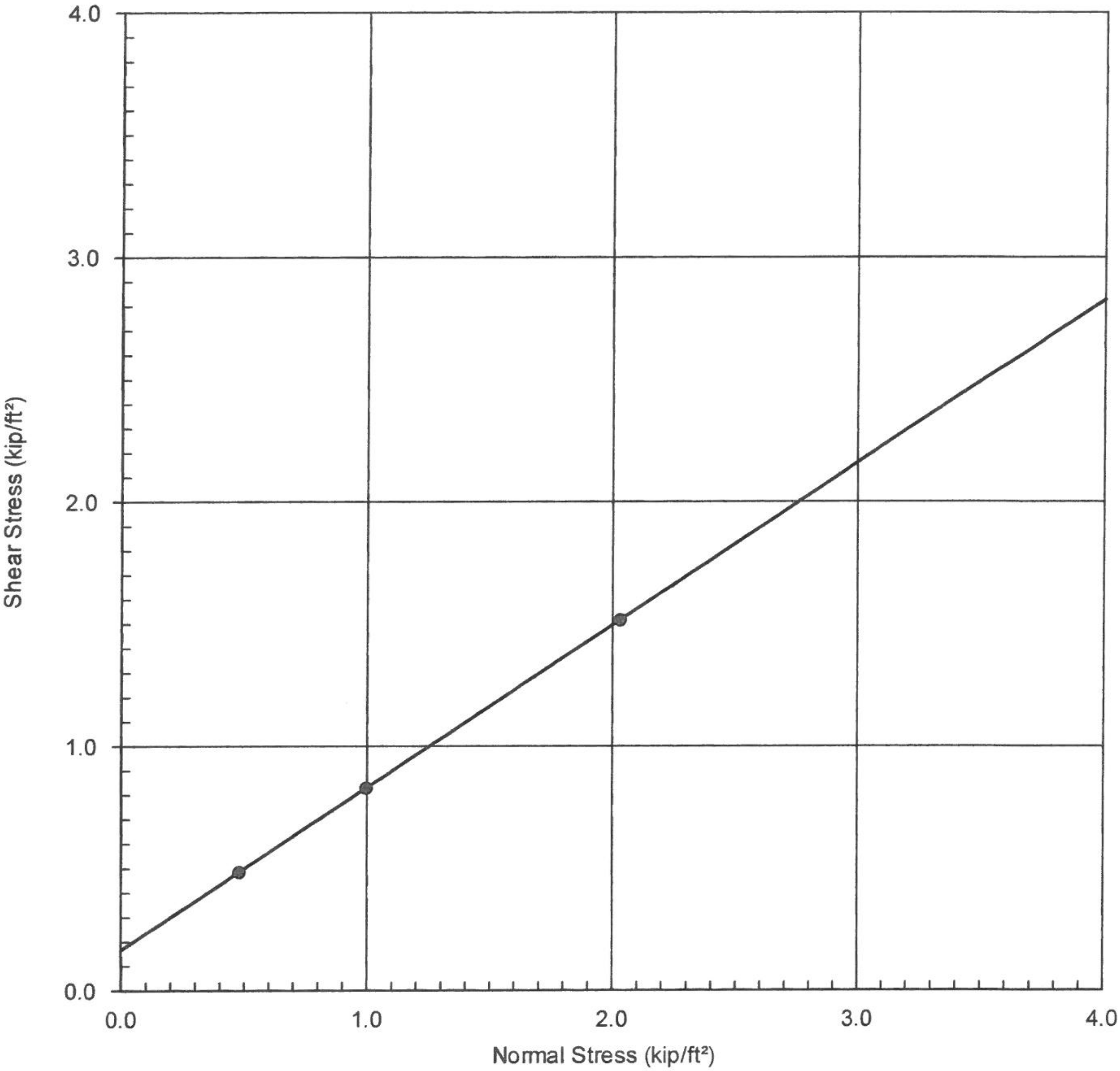


Boring No. : B-5
Depth (ft.) : 3.0
Sample Type : Relatively Undisturbed
Soil Type : Silty Clay with Fine Sand

Cohesion(C) = 775 psf
Friction (ϕ) = 20°
Dry Density (pcf) = 121.8
Moisture (%) = 11.1

Project Name: Monarch Bay Entrance/Park Improvements, Dana Point	Project No.: 6925.20
ASSOCIATED SOILS ENGINEERING, INC.	DIRECT SHEAR TEST RESULTS (ASTM D 3080)

PLATE D-3

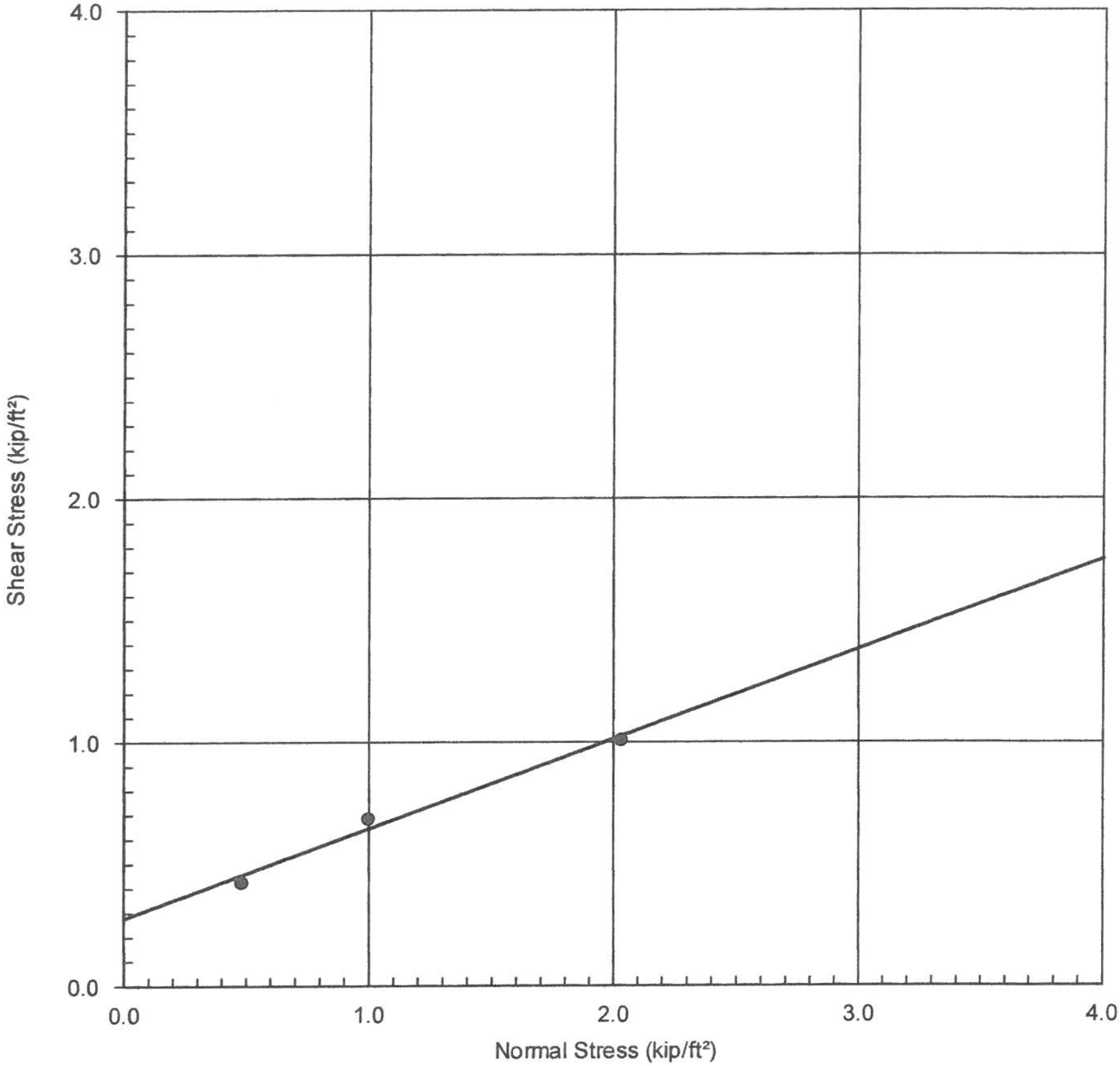


Boring No. : B-2
Depth (ft.) : 0.25-5.0
Sample Type : Remolded (90% of Maximum Density)
Soil Type : Silty Fine Sand

Cohesion(C) = 165 psf
Friction (ϕ) = 33.5°
Dry Density (pcf) = 107.6
Moisture (%) = 11.5

Project Name:Monarch Bay Entrance/Park Improvements,Dana Point	Project No.: 6925.20
ASSOCIATED SOILS ENGINEERING, INC.	DIRECT SHEAR TEST RESULTS (ASTM D 3080)

PLATE D-4



Boring No.	: B-5	Cohesion(C)	= 275 psf
Depth (ft.)	: 0.67-5.0	Friction (ϕ)	= 20°
Sample Type	: Remolded (90% of Maximum Density)	Dry Density (pcf)	= 113.9
Soil Type	: Fine Sandy Clay	Moisture (%)	= 10.0

Project Name:Monarch Bay Entrance/Park Improvements,Dana Point	Project No.: 6925.20
ASSOCIATED SOILS ENGINEERING, INC.	DIRECT SHEAR TEST RESULTS (ASTM D 3080)

PLATE D-5

APPENDIX B - SITE FAULTING/SEISMICITY DATA

Plates I-1 and I-2	EQFAULT – Deterministic Estimation of Peak Acceleration from Digitized Faults

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***********************
*                     *
*   E Q F A U L T     *
*                     *
*   Version 3.00      *
*                     *
***********************
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DETERMINISTIC ESTIMATION OF
PEAK ACCELERATION FROM DIGITIZED FAULTS

JOB NUMBER: 6925.20

DATE: 03-02-2020

JOB NAME: Monarch Bay Communiuty Entrance and Park Improvements
Monarch Bay DFrive & Pacific Coast Highway,Dana Point,CA
CALCULATION NAME: Test Run Analysis

FAULT-DATA-FILE NAME: C:\Program Files\EQFAULT1\Cgsflte.dat

SITE COORDINATES:
SITE LATITUDE: 33.4869
SITE LONGITUDE: 117.7280

SEARCH RADIUS: 62 mi

ATTENUATION RELATION: 20) Sadigh et al. (1997) Horiz. - Soil
UNCERTAINTY (M=Median, S=Sigma): M Number of Sigmas: 0.0
DISTANCE MEASURE: clodis
SCOND: 0
Basement Depth: 5.00 km Campbell SSR: Campbell SHR:
COMPUTE PEAK HORIZONTAL ACCELERATION

FAULT-DATA FILE USED: C:\Program Files\EQFAULT1\Cgsflte.dat

MINIMUM DEPTH VALUE (km): 0.0

EQFAULT SUMMARY

DETERMINISTIC SITE PARAMETERS

PLATE I–1

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
NEWPORT-INGLEWOOD (Offshore)	2.1(3.4)	7.1	0.471	X
SAN JOAQUIN HILLS	5.2(8.3)	6.6	0.401	X
NEWPORT-INGLEWOOD (L.A.Basin)	14.0(22.5)	7.1	0.196	VIII
PALOS VERDES	17.9(28.8)	7.3	0.176	VIII
CORONADO BANK	19.0(30.5)	7.6	0.194	VIII
ELSINORE (GLEN IVY)	22.6(36.3)	6.8	0.106	VII
CHINO-CENTRAL AVE. (Elsinore)	23.6(38.0)	6.7	0.121	VII
ELSINORE (TEMECULA)	24.4(39.3)	6.8	0.097	VII
WHITTIER	25.9(41.7)	6.8	0.091	VII
ROSE CANYON	30.7(49.4)	7.2	0.097	VII
PUENTE HILLS BLIND THRUST	31.6(50.8)	7.1	0.114	VII
SAN JOSE	39.1(63.0)	6.4	0.051	VI
ELSINORE (JULIAN)	41.9(67.4)	7.1	0.063	VI
SIERRA MADRE	44.0(70.8)	7.2	0.082	VII
CUCAMONGA	44.1(71.0)	6.9	0.065	VI
UPPER ELYSIAN PARK BLIND THRUST	45.5(73.3)	6.4	0.042	VI
SAN JACINTO-SAN JACINTO VALLEY	45.9(73.9)	6.9	0.048	VI
SAN JACINTO-SAN BERNARDINO	46.3(74.5)	6.7	0.041	V
RAYMOND	48.8(78.6)	6.5	0.042	V
SAN JACINTO-ANZA	49.9(80.3)	7.2	0.055	VI
CLAMSHELL-SAWPIT	50.2(80.8)	6.5	0.040	V
VERDUGO	50.8(81.8)	6.9	0.054	VI
HOLLYWOOD	52.4(84.3)	6.4	0.034	V
SAN ANDREAS - SB-Coach. M-1b-2	54.1(87.0)	7.7	0.071	VI
SAN ANDREAS - Whole M-1a	54.1(87.0)	8.0	0.088	VII
SAN ANDREAS - SB-Coach. M-2b	54.1(87.0)	7.7	0.071	VI
SAN ANDREAS - San Bernardino M-1	54.1(87.0)	7.5	0.062	VI
SANTA MONICA	56.2(90.5)	6.6	0.037	V
SAN ANDREAS - 1857 Rupture M-2a	57.0(91.8)	7.8	0.072	VI
SAN ANDREAS - Mojave M-1c-3	57.0(91.8)	7.4	0.053	VI
SAN ANDREAS - Cho-Moj M-1b-1	57.0(91.8)	7.8	0.072	VI
CLEGHORN	58.6(94.3)	6.5	0.025	V
MALIBU COAST	59.7(96.0)	6.7	0.037	V
NORTH FRONTAL FAULT ZONE (West)	60.5(97.4)	7.2	0.054	VI

-END OF SEARCH- 34 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE NEWPORT-INGLEWOOD (Offshore) FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 2.1 MILES (3.4 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.4708 g

PLATE I-2

APPENDIX D - LIST OF REFERENCES

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3. California Geological Survey, 2001, Seismic Hazard Zones Official Map, Dana Point Quadrangle, released December 21.

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5. California Building Code, 2019 Edition: Sacramento, CA, California Building Standards Commission, 2 Volumes.

6. Cao, T., Bryant, W.A. Rowshandel, B., Branum, D., and Wills, C.J., 2003, the Revised 2002 California Probabilistic Seismic Hazard Maps, California Geological Survey.

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8. Foundation and Earth Structures Design Manual 7.2 (Navfac DM-7.2), 1982, Department of the Navy, Naval Facilities Engineering Command, p. 347.

9. Stewart, J.P., Whang, D.H., Moyneur, M., and Duku, P., 2004, Seismic Compression of As-Compacted Fill Soils with Variable Level of Fines content and Fines Plasticity, CUREE Publication No. EDA-05, 101p.

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13. Building Code Requirements for Structural Concrete (ACI 318-19) and Commentary, 2019, published by American Concrete Institute, Farmington Hill, MI, 623p.

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15. Civil Engineering Pavements, Design Manual 5.4 (DM 5.4), 1979, Department of the Navy, Naval Facilities Engineering Command, 60 pp.

16. California Division of Mines and Geology (now California Geological Survey), Revised 1994, Fault Rupture Hazard Zones in California, Special Publication 42.

APPENDIX D - LIST OF REFERENCES - continued

17. California Division of Mines and Geology (now California Geological Survey), 1998, Map of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada, Published February.

18. Corrosion Guidelines, Version 2.0, November 2012, Published by California Department of Transportation (Caltrans), 44p.

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20. Legg, M.R., Borrero, J.C., and Synolakis, C.E., 2003, Evaluation of Tsunami Risk to Southern California Coastal Cities, The 2002 NEHRP Professional Fellowship Report, PF 2002-11, Earthquake Engineering Research Institute.

21. California Geological Survey, 2009, Tsunami Inundation Map for Emergency Planning, State of California, County of Orange, Dana Point Quadrangle/San Juan Capistrano Quadrangle, dated March 15.

22. Permeable Interlocking Concrete Pavements, Third Edition, 2006, published by Interlocking Concrete Pavement Institute (ICPI), 48 p.

23. Structural Design of Interlocking Concrete Pavement for Municipal Streets and Roadways, 2010, American Society of Civil Engineers, ASCE Standard 58-10.