TABLE OF CONTENTS

PAGE

TABLE OF CONTENTS (Continued)

PAGE

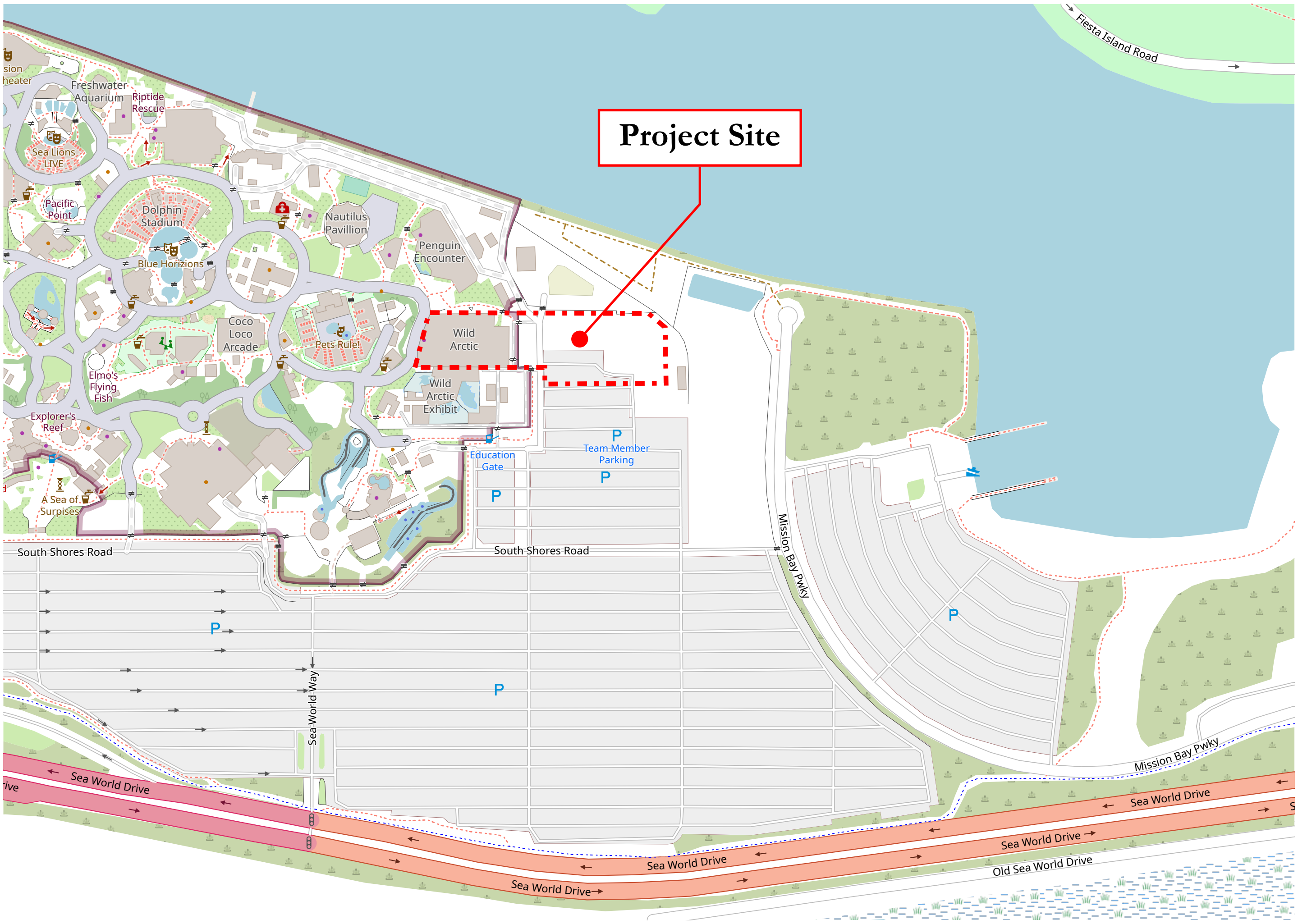
TABLES

FIGURES

PLATES

APPENDICES

SITE VICINITY



SEAWORLD 2021 PROJECT 500 SEAWORLD DRIVE, SAN DIEGO, CA	
DATE: SEPTEMBER 2019	REPORT NO.: 2190160.01
BY: SCC	FIGURE NO.: 1

CHRISTIAN WHEELER
ENGINEERING

plans should be submitted to Christian Wheeler Engineering for review to determine their conformance with our recommendations and to determine whether any additional subsurface investigation, laboratory testing and/or recommendations are necessary. Our professional services have been performed, our findings obtained and our recommendations prepared in accordance with generally accepted engineering principles and practices. This warranty is in lieu of all other warranties, express or implied.

SCOPE OF SERVICES

Our geotechnical investigation generally consisted of surface reconnaissance, subsurface exploration, review of previous subsurface explorations, analysis of the previous field data, and review of relevant readily available geologic literature. More specifically, our services included the following items.

- Obtaining a boring permit from the County of San Diego Department of Environmental Health to conduct the proposed subsurface investigation.
- Performing five Cone Penetrometer Tests (CPT) with a truck-mounted rig to explore the existing soil conditions. Shear wave velocity measurements were taken in one of the CPTs.
- Backfilling the CPT holes using a grout or a grout/bentonite mix as required by the County of San Diego Department of Environmental Health.
- Evaluating, by CPT data and our past experience with the park, the engineering properties of the various soil strata that may influence the proposed construction, including bearing capacities and settlement potential.
- Describing the general geology at the site, including possible geologic hazards that could have an effect on the proposed construction, and provide the seismic design parameters as required by the 2019 edition of the California Building Code.
- Addressing potential construction difficulties that may be encountered due to soil conditions, groundwater or geologic hazards, and provide geotechnical recommendations to deal with these difficulties.
- Quantitatively addressing the potential for soil liquefaction and dynamic settlement at the site in the event of a major, proximal seismic event.
- Providing site preparation and remedial grading recommendations for the anticipated work.
- Providing foundation recommendations for the type of construction anticipated and developing soil engineering design criteria for the recommended foundation designs.
- Providing this geotechnical report presenting the results of our evaluation, including a plot plan showing the locations of current and previous subsurface explorations, excavation logs, and our

conclusions and recommendations for the proposed project. The report will be provided as an electronic document in Portable Document Format (PDF).

It was not within the scope of our services to perform laboratory tests to evaluate the chemical characteristics of the on-site soils in regard to their potentially corrosive impact to on-grade concrete and below grade improvements. If requested, we can obtain and submit representative soil samples to a chemical laboratory for analysis; however, it should be understood that Christian Wheeler Engineering does not practice corrosion engineering. If such an analysis is necessary, we recommend that the client retain an engineering firm that specializes in this field to consult with them on this matter.

FINDINGS

SITE DESCRIPTION

The project site is located along the southern side of the Pacific Passage area of the Mission Bay Park, within the eastern portion of the SeaWorld lease-held property, in San Diego, California. The project site is generally located to the east of the existing Wild Arctic attraction and is currently a paved parking and storage area. The SeaWorld property and surrounding areas originally consisted of tidal flats that were reclaimed in the late 1950's by placing material dredged generated during the construction of Mission Bay as fill to raise the elevation. The specific project area has not previously supported structures. Topographically, the project area is relatively level with elevations generally ranging from about 17½ to 21½ feet based on the topographic map prepared by Rick Engineering (SeaWorld datum).

GENERAL GEOLOGY AND SUBSURFACE CONDITIONS

GEOLOGIC SETTING AND SOIL DESCRIPTION: The subject site is located in the Coastal Plains Physiographic Province of San Diego County. In order to evaluate the subsurface conditions at the project area, we have performed five Cone Penetration Tests at the subject site as well as reviewed previous subsurface explorations performed for the adjacent proposed 2020 project, and existing Journey to Atlantis and Wild Arctic attractions. Based on this information, our experience with the park, and our analysis of other readily available, pertinent geologic and geotechnical literature, we have determined that the site is underlain by artificial fill soils and estuarine deposits, which are underlain at depth by Quaternary-age paralic deposits. These materials are described below:

ARTIFICIAL FILL (Qaf): Mechanically- and hydraulically-placed fill materials are expected to extend to depths on the order of about 20 feet below the existing grades. Based on the similarities in composition and consistencies of these fill materials, no differentiation between mechanically- and hydraulically-placed fills is made in this report. In general, the upper three to five feet of fill materials consist of medium dense, moist, silty sand (SM) and poorly graded sand with silt (SP-SM). Below that depth, the fill materials typically consist of loose/soft, moist to saturated, sandy silt (ML) and silty sand (SM).

PARALIC ESTUARINE DEPOSITS (Qpe): Below the existing fill are Holocene-age estuarine deposits. The estuarine deposits consist of generally soft to medium stiff sandy silts (ML) and silty clays (CL) with interbeds of loose to medium dense silty sands (SM). These materials are below the water table and are saturated. Based on the results of our CPT soundings, the estuarine deposits are expected to extend to an approximate depth of 115 to 120 feet below the existing site grades.

OLD PARALIC DEPOSITS (Qop): The project area is underlain at depth by competent, Quaternary-age old paralic deposits. Though not observed, the old paralic deposits in the general site area typically consist of dense to very dense, clayey and silty sandstone and very stiff to hard, sandy claystone. Based on the results of previous CPT soundings, the old paralic deposits were encountered at a depth of 115 to 120 feet below the existing grades.

GROUNDWATER: Groundwater was measured in our Cone Penetration Tests at depths ranging from 15 to 18 feet below the existing grade. Using the topographic map by Rick Engineering, such depths correspond to elevations ranging from 2 to 5 feet. These elevations are consistent with our experience at the park, in which the local groundwater table is typically located at elevations ranging from about 2 feet to 6 feet (SeaWorld datum). However, it should be noted that the borings drilled for the adjacent Wild Artic attraction encountered groundwater at depths ranging from 9 to 15 feet below previously existing grades, which correlates to elevations of 5½ feet to 13½ feet (MSL datum per previous report). It should be noted that shallower zones of perched groundwater or wet to nearly saturated fine clays and silts are also common above the water table in the hydraulically-placed fills. Variations in subsurface water (including perched water zones and seepage) may result from fluctuations in the ground surface topography, subsurface stratification, precipitation, irrigation, and other factors that may not have been evident at the time of the investigation. It should also be recognized that minor groundwater seepage problems might occur after development of a site even where none were present before development. These are usually minor phenomena and are often the result of an alteration in drainage patterns and/or an increase in irrigation water. It is further our opinion that these problems can be most effectively corrected on an individual basis if and when they occur.

TECTONIC SETTING: It should be noted that much of Southern California, including the San Diego County area, is characterized by a series of Quaternary-age fault zones that consist of several individual, en echelon faults that generally strike in a northerly to northwesterly direction. Some of these fault zones (and the individual faults within the zones) are classified as "active" according to the criteria of the California Division of Mines and Geology. Active fault zones are those that have shown conclusive evidence of faulting during the Holocene Epoch (the most recent 11,000 years).

The Division of Mines and Geology used the term "potentially active" on Earthquake Fault Zone maps until 1988 to refer to all Quaternary-age faults for the purpose of evaluation for possible zonation in accordance with the Alquist-Priolo Earthquake Fault Zoning Act. The Alquist-Priolo Act requires the State Geologist to zone faults that are "sufficiently active" and "well-defined" to have a relatively high potential for ground rupture. The Division of Mines and Geology no longer uses the term "potentially active." However, the City of San Diego has elected to continue to use the term "potentially active" to refer to certain faults that demonstrated movement during the Pleistocene epoch (11,000 to 1.6 million years before the present) but that do not have substantiated Holocene movement. It should be recognized that the Alquist-Priolo Act (Division 2, Chapter 7.5, Section 2624) authorizes individual cities and counties to establish policies and criteria that are stricter than those established by the Alquist-Priolo Act.

A review of available geologic maps indicates that the active Rose Canyon Fault Zone is located approximately 1.5 miles to the east of the subject site. Other active fault zones in the region that could possibly affect the site include the Coronado Bank and San Clemente Fault Zones to the west, the offshore segment of the Newport-Inglewood and Palos Verdes Fault Zones to the northwest, and the Elsinore, Earthquake Valley, San Jacinto, and San Andreas Fault Zones to the northeast. The following Table I presents those proximal, active faults that are anticipated to most significantly contribute to the ground-motion hazard at the site.

TABLE I: PROXIMAL FAULT ZONES

Fault Zone	Distance
Rose Canyon	2.5 miles
Coronado Bank	10 miles
Newport-Inglewood	19 miles
Palos Verdes	38 miles
Elsinore	42 miles
San Clemente	46 miles
Earthquake Valley	47 miles
San Jacinto	62 miles
San Andreas	90 miles

GEOLOGIC HAZARDS

GEOLOGIC HAZARDS CATEGORY: The site is located in Geologic Hazard Category 31 according to the City of San Diego Seismic Safety Study (Sheet 20). Hazard Category 31 is assigned to areas characterized as having a high potential for soil liquefaction in the event of a major seismic event. Such characterization is based on the relatively shallow groundwater table and presence of hydraulic fills and other soft, cohesionless sediments within the area of the subject site. Discussion of the geologic hazards associated with seismically induced soil liquefaction at the subject site is presented in the Liquefaction and Lateral Ground Spreading sections of this report.

LANDSLIDE POTENTIAL AND SLOPE STABILITY: As part of this investigation we reviewed the publication, "Landslide Hazards in the Southern Part of the San Diego Metropolitan Area" by Tan, 1995. This reference is a comprehensive study that classifies San Diego County into areas of relative landslide susceptibility. The subject site is located in Area 1. Land within Area 1 is considered to be the least susceptible to slope failures. Based on the absence of significant slopes within the vicinity of the subject site, the potential for slope failures can be considered negligible.

SEISMIC HAZARD: A likely geologic hazard to affect the site is ground shaking as a result of movement along one of the major active fault zones mentioned in the "Tectonic Setting" section of this report. Seismic design parameters were determined in accordance with Chapter 16 of the *2019 California Building Code (CBC)* and the applicable sections of *ASCE/SEI 7-16 Minimum Design Loads and Associated Criteria for Buildings and Other Structures*. For the subject site, shear wave velocities measured in our CPT-1 indicate that the upper 100 feet of geologic subgrade has a V_{S30} value of 720 feet per second and can be characterized as Soil Site Class D.

It can be noted that sites underlain by liquefaction-susceptible soils should be designated as Soil Site Class F, requiring a site response analysis. However, as discussed in Section 20.3.1 of ASCE/SEI 7-16, for structures having fundamental periods of vibration equal to or less than 0.5 second, it is not required to perform a site response analysis. We understand that the proposed structures will have fundamental periods less than 0.5 second and can therefore be designed using Soil Site Class D as described above.

In accordance with Section 11.4.8 of ASCE/SEI 7-16, structures on Soil Site Class D or E sites that have a mapped MCE_R spectral response acceleration parameter (S_1) value greater than or equal to 0.2 require a site-specific ground motion hazard analysis or the seismic response coefficient (C_S) must be adjusted to adequately characterize the site response (Exception 2). The following Table II presents the seismic design parameters based on Exception 2 in Section 11.4.8.

TABLE II: CBC 2019/ASCE 7-16 – SEISMIC DESIGN PARAMETERS

CBC – Chapter 16 Section	Seismic Design Parameter	Recommended Value
Section 1613.2.2	Soil Site Class	D
Figure 1613.2.1 (1)	MCE_R Acceleration for Short Periods (0.2 sec), S_s	1.391 g
Figure 1613.2.1 (2)	MCE_R Acceleration for 1.0 Sec Periods (1.0 sec), S_1	0.480 g
Table 1613.2.3 (1)	Site Coefficient, F_a	1.000
Table 1613.3.3 (2)	Site Coefficient, F_v	1.820
Section 1613.2.3	S_{MS} = MCE_R Spectral Response at 0.2 sec. = $(S_s)(F_a)$	1.391 g
Section 1613.2.3	S_{M1} = MCE_R Spectral Response at 1.0 sec. = $(S_1)(F_v)$	0.874 g
Section 1613.2.4	S_{DS} = Design Spectral Response at 0.2 sec. = 2/3(S_{MS})	0.927 g
Section 1613.2.4	S_{D1} = Design Spectral Response at 1.0 sec. = 2/3(S_{M1})	0.582 g
Section 1613.2.5	Seismic Design Category	D
ASCE 7-16 Fig. 22-14	Mapped Long-Period Transition Period, T_L	8 sec
ASCE 7-16 Eq 12.8-3	Seismic Response Coefficient, C_S	Multiply by 1.5
Section 1803.2.12	PGA_M per Section 11.8.3 of ASCE 7	0.70 g

FLOODING: As delineated on the Flood Insurance Rate Map (Panel 1613F) prepared by the Federal Emergency Management Agency, the site is located within an area labeled as “Area of Minimal Flooding-Zone X.”

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

Portions of Mission Bay and surrounding areas are located within an area that may be affected by tsunamis. The Multi-Jurisdictional Hazard Mitigation Plan of the County of San Diego (URS, 2010) maps the project site outside of the area susceptible to the maximum projected runup from a tsunami. Additionally, as presented on the La Jolla Quadrangle of the State’s Tsunami Inundation Map for Emergency Planning (June, 2009), the subject site is outside of the area of anticipated tsunami runup. Furthermore, although the Mission Bay area is within an area that may be affected by tsunamis, based on the bathymetry of the bay and offshore San Diego coastline, the fact that historical tsunamis reaching San Diego have generally been well within the normal tidal range, the distance from the site to the entrance to Mission Bay, and the elevation of the site of ±23-25 feet, it is our professional opinion and judgment that tsunami hazard at the site is relatively low and no greater at the subject site than it is along the adjacent areas of the entertainment park and other portions of Mission Bay that are not in close proximity to the bay's entrance or the barrier beach area along the west side of Mission Beach.

SEICHES: Seiches are periodic oscillations in large bodies of water such as lakes, harbors, bays or reservoirs. Although the site is located adjacent to Mission Bay, due to the size and configuration of Mission Bay, it is our opinion that the risk potential for damage caused by seiches is relatively low.

LIQUEFACTION

GENERAL: The subject site is in an area considered susceptible to liquefaction. In order to be subject to liquefaction, three conditions must be present: loose sandy or cohesionless silty deposits, shallow groundwater, and earthquake shaking of sufficient magnitude and duration. Based on our site-specific study, it appears that shallow groundwater is present at the site and strong earthquake shaking may affect the site. Additionally, as described in the Geologic Setting and Soil Description section of this report above, the materials below the shallow water table in the project area consist of Holocene-age fill material and estuarine deposits that contain layers of sand, silty sand, and low to medium plasticity silts (ML) that are expected to have soil properties conducive to liquefaction.

It should be noted that the following discussion is in no way a guarantee that the analysis will accurately predict the liquefaction potential at the site. The analysis provides general information only on the site liquefaction potential. It should be noted that many of the parameters used in liquefaction evaluations are subjective and open to interpretation, and that much is yet unknown about both the seismicity of the San Diego area and the phenomenon of liquefaction.

DESCRIPTION OF ANALYSIS: Our analysis was performed using the Cliq (version 3.0) software developed by Geologismiki, in which the results of our CPT soundings were input and evaluated in accordance with the procedure recommended by the National Center For Earthquake Engineering Research (NCEER, 1998). Our analyses were limited to the upper 60 feet of the existing soils below the proposed improvements. Liquefaction of soils at greater depths is expected to be less likely based on the age of the deeper soils. Additionally, an algorithm was applied within the software to make corrections for thin stiff layers embedded within softer zones (Robertson, 2009).

EARTHQUAKE PARAMETERS: As permitted in Section 1803.5.12 of the California Building Code, our calculations were performed using a peak ground acceleration (PGA_M = 0.70g) as determined using the procedures set forth in Section 11.8.3 of ASCE 7-16. We have also performed a seismic hazard deaggregation using the interactive program available on the U. S. Geological Survey website. Within the USGS program, the site coordinates were entered and a deaggregation was performed based on the peak ground acceleration with one percent probability of exceedance in 50 years (0.78g) for soil with Vs^{30} = 260 m/s (Soil Site Class D). For the subject site, this yielded a modal earthquake magnitude of 6.9. Based on this result and the proximity of the site to the Rose Canyon (7.2 Magnitude) and Coronado Bank (7.6 Magnitude) Fault Zones, this result was used in our analyses.

POTENTIAL FOR LIQUEFACTION: Using the parameters described above, the results of our liquefaction analyses indicate that much of the saturated sandy and silty portions of the estuarine deposits within the upper 60 feet of soil below the proposed improvements possess factors-of-safety against soil liquefaction of less than 1.0 and are therefore considered potentially liquefiable. A complete report of our analysis is presented in Appendix C of this report.

POST LIQUEFACTION RECONSOLIDATION SETTLEMENT: The potential amount of total vertical settlement due to reconsolidation of the liquefied soils was estimated within the Cliq software using the methods presented by Zhang et al, 2002 with a depth-weighted dynamic settlement profile (Cetin et al, 2009) as recommended by Dr. Peter Robertson (2014). The estimated settlement within our five CPTs ranged from about 1 to 3 inches, with an average of nearly 2 inches. It can be noted that, for sites with relatively small lateral displacement (i.e. less than one foot), predicted settlements are typically within a factor of two relative to those observed (Seed et al, 2003).

In terms of differential settlement, CGS Special Publication 117 notes that considerable difficulty exists in trying to “reliably estimate” the amount of differential settlement at a site caused by soil liquefaction. As such, a conservative estimate of differential settlement at any given site can be assumed to be two-thirds of the total liquefaction-induced settlement (CGS, 2008). Using this criterion, without any deep ground modification, the subject project area may be assumed to be subject to approximately 2 inches of liquefaction-induced, differential settlement. This estimated differential settlement can be assumed to occur over a horizontal distance of 40 feet, which equates to an angular distortion of 0.0041L.

LATERAL SPREADING: Lateral ground spreading can occur when viscous liquefied soils flow downslope, usually towards a river channel or shoreline. Within the referenced Cliq software, the results of our liquefaction analyses were used in conjunction with the methodology developed by Zhang, Robertson, and Brachman (2004) to estimate the value of lateral spreading for a gently sloping ground surface with no free face. Based on the results of our analyses, the estimated lateral spreading value for the design earthquake conditions is between approximately 3 to 9 inches.

CONCLUSIONS

In general, it is our opinion that the subject site is suitable to support the proposed coaster attraction provided the geotechnical design and construction criteria presented in the following section are followed. Based on our investigation, we offer the conclusions listed below.

- The main geotechnical and geologic conditions that will impact the proposed construction are the presence of fill material at the anticipated foundation level with marginal strength characteristics and the potential for liquefaction of the underlying soils during a major seismic event.
- The structural engineer has evaluated using either shallow or deep foundation systems to support the coaster based on our preliminary geotechnical criteria. We understand that, based on the relatively small loads imparted by the coaster, shallow foundations are feasible. However, we understand that deeper foundations, such as augercast piles, may be needed based on the presence of underground improvements that cannot be relocated.
- The Guidelines for Evaluating and Mitigating Seismic Hazards in California (CGS, Special Publication 117A) indicates that for liquefiable sites "the minimum level of mitigation for a project should be to reduce the risk of ground failure during an earthquake to a level that does not cause the collapse of buildings for human occupancy, but in most cases not to a level of no ground failure at all." Based on our discussions with the coaster engineer from Intamin Amusement Rides, given the estimated differential settlement of 2 inches over 40 feet (0.0041L), the coaster structure is able to meet this standard and provide a life-safety performance level. Additionally, this estimated value of differential settlement does not exceed the threshold for "Other single-story structures" in Risk Category II supported on shallow foundations as noted in Table 12.3-3 in ASCE/SEI 7-16. Such acceptance, however, does preclude the possibility of some structural damage and settlement occurring as a result of a major seismic event to the extent that the coaster and associated buildings may need extensive repair or replacement.
- Where the planned structures are to be supported by shallow foundations, it will be necessary to perform remedial grading in the form of overexcavation and recompaction of the foundation soil. Additionally, pad footings should be connected via tie beams per ASCE/SEI 7-16.

RECOMMENDATIONS

GRADING AND EARTHWORK

GENERAL: All grading should conform to the guidelines presented in Appendix J of the California Building Code, the minimum requirements of the City of San Diego, and the following recommendations. Prior to grading, a representative of Christian Wheeler Engineering should be present at the preconstruction meeting to provide additional grading guidelines, if necessary, and to review the earthwork schedule.

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

CLEARING AND GRUBBING: Site grading should begin with the removal of all existing structures, asphalt and concrete pavement, trees, light poles and underground utilities in the portions of site that will be graded and/or will receive new improvements. Crushed asphalt and/or concrete can be incorporated into fills from a geotechnical standpoint provided the material meets the requirements in the "Compaction and Method of Filling" section of this report. The other removed materials should be disposed of off-site. Any abandoned underground pipes found during the grading operation should be removed and the resulting depressions backfilled with uniformly compacted fill material.

EXCAVATION CHARACTERISTICS: We expect that planned excavations and excavations for the removal of unsuitable soils can be made using normal grading equipment. However, it should be recognized that that some very fat, highly plastic clays or saturated silts are sometimes encountered above the water table. These materials will not support larger grading and may require special equipment and/or temporary stabilization. Additionally, the clay is not suitable for use as structural fill material. Other saturated soils may be used as fill provided they are dried back to a suitable moisture content (see "Compaction and Method of Filling" section). Also, it is not uncommon to find some buried debris in excavations. Any such material found that is determined by the geotechnical engineer to be unsuitable for fill material will also need to be disposed of off-site.

SITE PREPARATION: Specific site preparation recommendations for different elements of the proposed attraction are presented below. No special site preparation is necessary at this time where augercast piles are used. All fill should be placed in accordance with the "Compaction and Method of Filling" section of this report.

- **Structures:** For structures that are supported by shallow foundations, we recommend that a mat of properly compacted fill be constructed below the shallow foundations. This mat should extend at least six feet below the bottom of the footing for the coaster and four feet below the bottom of the footing for other structures. Where possible, the mat should extend at least five feet outside the perimeter foundation. The recommended fill mat can be constructed by removing the existing soil and placing it back as properly compacted fill; however, prior to placing fill the bottom of the excavation should be approved by a member of our engineering or geology staff. If soft, pumping, or otherwise unsuitable soils are exposed at the removal bottom that cannot be properly compacted, it will be necessary to stabilize the bottom soils prior to placing structural fill (see Processing of Fill Areas).

- **Exterior Flatwork Areas:** In areas that will receive on-grade concrete flatwork, the subgrade soils should be scarified to a depth of at least 12 inches, moisture conditioned, and compacted in place prior to placing concrete. If soils considered to be unsuitable to support the flatwork are exposed in the subgrade, removals may be necessary. The depth of removal will need to be determined by our firm when such conditions are exposed.

PROCESSING OF FILL AREAS: Prior to placing any new fill soils or constructing any new improvements in areas that have been cleaned out and approved to receive fill, the exposed soils should be scarified to a depth of 12 inches, moisture-conditioned, and compacted to at least 90 percent relative compaction. If soft, pumping, or otherwise unsuitable soils are exposed at the removal bottom that cannot be properly compacted, it will be necessary to stabilize the bottom soils prior to placing structural fill. One method is to construct a stabilization blanket consisting of at least one foot of ¾-inch crushed rock wrapped entirely in stabilization/filter fabric such as Mirafi 600X (or equivalent). Adjoining fabric panels should be overlapped at least 12 inches. Depending on the degree of pumping, an alternative method of stabilization may consist of placing 2 to 3 layers of stabilization fabric, such as Mirafi HP570 (or equivalent), or grid material, such as Mirafi BX1100 (or equivalent), within the lower fill. The initial layer would be placed at the excavation bottom and subsequent layers would be placed at 1 foot vertical intervals. Adjoining panels should be overlapped at least 6 inches.

DEWATERING: Based on the expected construction and the above recommendations, we expect that the excavations for the proposed structures will be above the local water table; however, the excavations may encounter perched groundwater and/or very wet to saturated, fine silts. In this case, it will likely be necessary to perform localized dewatering during construction to remove water from the excavation. If excavations for deep utilities extend below the water table, we recommend that a contractor specializing in construction dewatering be retained to design and perform the necessary dewatering. It is recommended that such dewatering be performed as much as possible on a localized basis in order to minimize its impact on adjacent improvements.

COMPACTION AND METHOD OF FILLING: All structural fill placed at the site should be compacted to a relative compaction of at least 90 percent of its maximum dry density as determined by ASTM Laboratory Test D1557. Fills should be placed at or slightly above optimum moisture content, in lifts six to eight inches thick, with each lift compacted by mechanical means. Fills should consist of approved earth material, free of trash or debris, roots, vegetation, or other materials determined to be unsuitable by our soil technicians or project geologist. Fill material should be free of rocks or lumps of soil in excess of twelve inches in maximum dimension. However, in the upper five feet of pad grade, no rocks or lumps of soil in excess of six inches should be allowed. All utility trench backfill should be compacted to a minimum of 90 percent of its maximum dry density.

TEMPORARY SLOPES: The contractor is solely responsible for designing and constructing stable, temporary excavations and will need to shore, slope, or bench the sides of trench excavations as required to maintain the stability of the excavation sides. The contractor's "competent person", as defined in the OSHA Construction Standards for Excavations, 29 CFR, Part 1926, should evaluate the soil exposed in the excavations as part of the contractor's safety process. We anticipate that the existing on-site soils will consist of Type C material. Our firm should be contacted to observe all temporary cut slopes during grading to ascertain that no unforeseen adverse conditions exist. No surcharge loads such as foundation loads, or soil or equipment stockpiles, vehicles, etc. should be allowed within a distance from the top of temporary slopes equal to half the slope height.

SURFACE DRAINAGE: The ground around the proposed structures should be graded so that surface water flows rapidly away from the structure without ponding. In general, we recommend that the ground adjacent to structure slope away at a gradient of at least 5 percent for a minimum distance of 10 feet. If the minimum distance of 10 feet cannot be achieved, an alternative method of drainage runoff away from the building at the termination of the 5 percent slope will need to be used. Swales and impervious surfaces that are located within 10 feet of the building should have a minimum slope of 2 percent. Rain gutters with downspouts that discharge runoff away from the structure into controlled drainage devices are also recommended.

GRADING PLAN REVIEW: The final grading plans should be submitted to this office for review in order to ascertain that the geotechnical recommendations remain applicable to the final plan and that no additional recommendations are needed due to changes in the anticipated development. Our firm should be notified of changes to the proposed project that could necessitate revisions of or additions to the information contained herein.

SHALLOW FOUNDATIONS

GENERAL: Provided the above site preparation is performed, it is our opinion that the proposed structures can be supported by shallow foundation systems. We recommend that individual pad footings or widely-spaced continuous footings be connected with intersecting tie beams as discussed below in order to help mitigate the possible effects of liquefaction-induced differential settlement. The following design recommendations are considered the minimum based on anticipated soil conditions and are not intended to be lieu of structural considerations.

DIMENSIONS: New continuous and individual pad footings should have a minimum embedment of 24 inches below finish grade. Individual footings or widely-spaced continuous footings should be connected using tie beams per ASCE/SEI 7-16. Footing widths and the associated allowable bearing pressure can be determined using Table III based on the allowable settlement. Intermediate values can be determined by interpolation. The allowable bearing pressures are applicable to all load combinations using an allowable stress design including those with wind and seismic.

TABLE III: SHALLOW FOUNDATION WIDTHS AND ALLOWABLE BEARING PRESSURES

Footing Width (ft)	¼-inch Settlement	½-inch Settlement	1-inch Settlement
5	1,300 psf	2,500 psf	3,000
8	1,000 psf	2,000 psf	2,400
12	900 psf	1,600 psf	1,800
15	750 psf	1,250 psf	1,500

SPRING STIFFNESS: The spring stiffness can be determined based on the Method 1 equations presented in ASCE 41-13 and/or FEMA 356. The soil criteria for such evaluations are presented in Table IV. The poisson ratio was assumed to be 0.25 and the effective shear modulus was determined based on an average initial shear modulus determined from cone penetration tests and the effective shear modulus ratio given in Table 8-2 from ASCE 41-13.

TABLE IV: SOIL PROPERTIES FOR SPRING STIFFNESS

Design Level	PGA (g)	Soil Site Class	Shear Ratio G/Go	Poisson Ratio, v	Initial Shear, Go (ksf)	Effective Shear, G (ksf)
PGA_M	0.70	D	0.2	0.25	1,800	360

FOOTING REINFORCING: Reinforcement requirements for foundations should be provided by the project structural engineer. However, we recommend that the minimum reinforcing for continuous footings consist of at least four No. 5 steel reinforcing bars, with two No. 5 bars positioned near the bottom of the footing and two No. 5 bars positioned near the top of the footing.

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

SETTLEMENT CHARACTERISTICS: The anticipated foundation settlement for the static condition are provided in Table III. It should be recognized that minor cracks normally occur in concrete slabs and foundations due to shrinkage during curing or redistribution of stresses, therefore some cracks should be anticipated. Such cracks are not necessarily an indication of excessive vertical movements. The estimated total and differential settlement in the event of liquefaction due to a design level earthquake event is 3 inches and 2 inches over a horizontal distance of 40 feet, respectively.

EXPANSIVE CHARACTERISTICS: Due to the generally "low" expansive potential of the on-site soils, special foundation design recommendations for heaving soils are considered unnecessary.

FOUNDATION EXCAVATION OBSERVATION: All footing excavations should be observed by Christian Wheeler Engineering prior to placing formwork or reinforcing steel to determine if the foundation recommendations presented herein are followed and that the foundation soils are as anticipated in the preparation of this report. All footing excavations should be excavated neat, level, and square. All loose or unsuitable material should be removed prior to the placement of concrete.

AUGERCAST PILES

GENERAL: We understand that 18-inch diameter augercast piles may be used to support the coaster. The project structural engineer should design all pile locations, dimensions, and reinforcing using the recommendations and design parameters presented below. The following foundation design criteria are intended to meet the maximum total and differential settlements requirements under typical service load conditions. Based on our discussions with the coaster engineer from Intamin, the coaster structure is able to provide a life-safety performance level given the estimated differential settlement of 2 inches in the event of soil liquefaction. The following geotechnical design criteria are not intended to limit the displacement of the structure in the event of liquefaction.

AXIAL CAPACITY: Based on the relatively high displacements that are necessary to mobilize end bearing resistance and the given settlement criteria, we recommend that the piles be designed based solely on frictional resistance. For dead plus live load conditions, the ultimate frictional resistance along the side of the pile can be taken as 1,000 pounds per square foot (psf). Assuming a safety factor of 2, the allowable frictional resistance can be taken as 500 psf. The allowable resistance may be increased by one-third when considering temporary loads such as wind. For earthquake loads, we recommend an ultimate frictional resistance 750 psf be used.

LPILE SOIL PARAMETERS: We understand that the project structural engineer will evaluate the lateral capacity of the augercast piles using the computer program LPILE. Table V provides a summary of the recommended soil parameters to be used in the evaluation.

TABLE V: LPILE SOIL PARAMETERS

Elevation Interval (ft)	Depth (ft)	Soil Type	Unit Weight (pci)	Undrained Shear Strength (psi)	Friction Angle (degrees)
17 to 7	0 to 10	Sand	0.0069	--	34
7 to -1	10 to 18	Stiff Clay without Free Water	0.0069	7	--
-1 to -7	18 to 24	Stiff Clay with Free Water	0.0033	9	--
-7 to -20	24 to 37	Sand	0.0033	--	38
-20 to -25	37 to 42	Stiff Clay with Free Water	0.0033	4	--
-25 to -43	42 to 60	Sand	0.0033	--	32

Note: Program default values can be used for soil input parameters ε_{50} and k.

TABLE VI: LPILE P-MULTIPLIER FOR GROUP EFFECTS

Center-to-Center Spacing in Pile Diameters	p-multiplier		
	First (Leading) Row	Second Row	Third Row
6	0.90	0.90	0.80
5	0.85	0.80	0.70
4	0.80	0.70	0.60
3	0.75	0.55	0.40

Group effects can be neglected for piles with a center-to-center spacing equal to 7 pile diameters or greater. For pile groups with center-to-center spacing closer than 7 piles diameters, a p-multiplier should be applied in the direction of loading to the p-y curves as shown in Table VI above.

AUGERCAST PILE CONSTRUCTION CONSIDERATIONS: The performance of auger-cast piles is dependent to a great extent on proper installation technique. We recommend that a contractor familiar and experienced with the installation of augercast piles be retained on the project. The following items should be considered during the construction of auger-cast piles:

- The rate of drilling penetration and rotation should be maintained at a level such that the auger is advanced without excessive mining of the soil along the pile sides.
- Once the required tip elevation is reached, grouting should begin immediately. The initial lift to blow the plug should limited to six inches (150 mm) in order to minimize potential stress relief at the bearing surface.
- After the initial lift, the grout should be pumped with sufficient pressure and the auger withdrawn slowly enough to maintain the hole and allow lateral penetration of the grout into soft or porous zones of surrounding soil. For the lowest 3 to 6 feet (0.9 to 1.8 meters) of the hole, the delivered grout volume should be approximately 200 percent of the theoretical volume required to fill the pile for that length. For the remainder of the pile, the delivered grout volume should be at least 120 percent of the theoretical volume.
- The grout pressure and auger withdrawal rate should be maintained at steady levels in order to construct a pile of uniform diameter without "necking".
- The grout should include additives that control setting and shrinkage, and must be fluid enough to be pumped easily without excessive pressure losses.

- All reinforcement should be inserted before the grout sets up, normally within ten minutes after the augers are withdrawn. The reinforcement should be placed in the center of the pile, extend the full length of the pile, and be plumb to avoid having it protrude from the grout into the soil.

MONITORING: The project geotechnical engineer should provide full-time observation and testing of the pile installation. Observations will include review of drill rates and injection pressures as well as the grout volumes placed, all of which should be included in the contractor's logs in terms of units per depth (maximum of 3-foot intervals). Tests will include those to quantify the pertinent physical properties of the grout placed, such as flow and compressive strength.

Prior to construction of the test pile (see below), we recommend that the piling contractor prepare and submit a pile installation plan that provides the items listed below.

- The proposed equipment (including sizes) to be used.
- A step-by-step description of the installation procedure.
- Target drilling and grouting parameters for pile installation, including auger rotation speed, drilling penetration rates, torque, applied crowd pressures, grout pressures, and grout volume factors.
- Details of methods of reinforcement placement.
- Mix designs for all grout to be used.
- Equipment and procedures for monitoring and recording auger rotation speed, auger penetration rates, auger depths, crowd pressure, grout pressure, and grout volumes during installation.

TESTING PROGRAM: We recommend that at least one test pile for each pile type be installed with monitoring by the Geotechnical Consultant to evaluate the suitability of the contractor's installation procedures and equipment, as well as our design assumptions. We recommend the maximum test load be two times the design load. Based on the subsurface conditions encountered, we recommend using the "Quick Load Test Method" referenced in ASTM D1143. We recommend the 100 percent test load application be held and monitored for a period of four hours. If reaction piles are used for applying the test loads, a portion of the reaction piles installed should be similar to the test pile (i.e. augercast piles) to aid in the installation evaluation. The test pile can be used as a production pile as long as the net "set" experienced during the load tests is in acceptable ranges.

FOUNDATION PLAN REVIEW

The final foundation plan and accompanying details and notes should be submitted to this office for review. The intent of our review will be to verify that the plans used for construction reflect the minimum dimensioning criteria presented in this section and that no additional criteria are required due to changes in the foundation type or layout. It is not our intent to review structural plans, notes, details, or calculations to verify that the design engineer has correctly applied the geotechnical design values. It is the responsibility of the design engineer to properly design/specify the foundations and other structural elements based on the requirements of the structure and considering the information presented in this report.

ON-GRADE SLABS

INTERIOR SLABS: We recommend that the interior slab-on-grade floors for the buildings be at least five inches thick. Interior slabs should be reinforced with at least No. 3 bars spaced at least 18 inches on center each way. The owner and the project structural engineer should determine if the on-grade slabs need to be designed for special loading conditions. For such cases, a subgrade modulus of 150 pounds per cubic inch can be assumed for the subgrade provided it is prepared as recommended in this report.

UNDER-SLAB VAPOR RETARDERS: Where floor coverings are installed, steps should be taken to minimize the transmission of moisture vapor from the subsoil through the interior slabs where it can potentially damage the interior floor coverings. We recommend that the owner/contractor follow national standards for the installation of vapor retarders below interior slabs as presented in currently published standards including ACI 302, "Guide to Concrete Floor and Slab Construction" and ASTM E1643, "Standard Practice for Installation of Water Vapor Retarder Used in Contact with Earth or Granular Fill Under Concrete Slabs".

EXTERIOR CONCRETE FLATWORK: Exterior slabs not subject to vehicular traffic should have a minimum thickness of 4 inches. Slabs that will be support vehicular traffic should have a minimum thickness of 6 inches. Reinforcement can be placed in exterior concrete flatwork to reduce the potential for cracking and movement. Control joints should be placed in exterior concrete flatwork to help control the location of shrinkage cracks. Spacing of control joints should be in accordance with the American Concrete Institute specifications.

Special attention should be paid to the method of concrete curing to reduce the potential for excessive shrinkage and resultant random cracking. It should be recognized that minor cracks occur normally in concrete slabs due to shrinkage. Some shrinkage cracks should be expected and are not necessarily an indication of excessive movement or structural distress.

EARTH RETAINING WALLS

PASSIVE PRESSURE: The passive pressure for the prevailing soil conditions may be considered to be 350 pounds per square foot per foot of depth for foundations in fill soil. This pressure may be increased one-third for seismic loading. The coefficient of friction for concrete to soil may be assumed to be 0.35 for the resistance to lateral movement. When combining frictional and passive resistance, the friction should be reduced by one-third. The upper 12 inches of exterior retaining wall footings should not be included in passive pressure calculations where abutted by landscaped areas.

ACTIVE PRESSURE: The active soil pressure for the design of unrestrained and restrained earth retaining structures with level backfill surface may be assumed to be equivalent to the pressure of a fluid weighing 35 and 55 pounds per cubic foot, respectively. Seismic pressure may be assumed to be equivalent to the pressure of a fluid weighing 15 pounds per cubic foot. In the case that the retaining wall is restrained at the top, the seismic pressure should be added only to the unrestrained value (35 pcf). Thirty percent of any area surcharge placed adjacent to the retaining wall may be assumed to act as a uniform horizontal pressure against the wall. If any other loads are anticipated, the Geotechnical Consultant should be contacted for the necessary increase in soil pressure. All values are based on a drained backfill condition.

WATERPROOFING AND SUBDRAINS: The project architect should provide (or coordinate) waterproofing details for the retaining walls. The design values presented above are based on a drained backfill condition and do not consider hydrostatic pressures. Unless hydrostatic pressures are incorporated into the design, the retaining wall designer should provide a subdrain detail. A typical retaining wall subdrain detail is presented as Plate No. 2 of this report. Additionally, outlets points for the retaining wall subdrains should be coordinated by the project civil engineer. For subterranean walls, it may be necessary to collect the subdrain water in sumps and then pump it to an appropriate outlet.

BACKFILL: All backfill soils should be compacted to at least 90 percent relative compaction. Expansive or clayey soils should not be used for backfill material. The wall should not be backfilled until the masonry has reached an adequate strength.

PRELIMINARY PAVEMENT SECTIONS

GENERAL: We expect that new asphalt concrete pavement will be installed for a new access road. The pavement sections provided in Table VII should be considered preliminary and should be used for planning purposes only. Final pavement designs should be determined after R-value tests have been performed in the actual subgrade material in place after grading. Presuming the grading recommendations presented previously are followed, we estimate that the subgrade soils will have an R-Value of at least 25. The Traffic Index and Traffic Categories shown below are assumed. The project client and/or civil engineer should determine whether these assumed values are appropriate for the traffic conditions.

ASPHALT CONCRETE: We expect that the access drive will primarily support passenger vehicles with only occasional heavily loaded vehicles such as delivery trucks and/or emergency vehicles with a maximum weight of 95,000 pounds. The asphalt concrete pavement section was calculated using the Caltrans design method using an assumed Traffic Index of 4.5.

TABLE VII: ASPHALT CONCRETE PAVEMENT SECTIONS

Pavement Type	Traffic Index	Pavement Thickness	Base Thickness	Base Material	Subgrade Compaction
Asphalt Concrete					
Access Road	4.5	3.0 in.	5.0 in.	CAB or Class II	95% in upper 12”
Access Road	4.5	3.0 in.	6.0 in.	CMB, PMB, Milled AC	95% in upper 12”

Remedial grading under the proposed pavement areas should be performed in accordance with the Site Preparation section of this report. Prior to placing the base material beneath asphalt concrete pavements, the subgrade soil should be scarified to a depth of 12 inches and compacted to at least 95 percent of its maximum dry density at a moisture content near optimum.

The base material could consist of Crushed Aggregate Base (CAB) or Class II Aggregate Base. The Crushed Aggregate Base should conform to the requirements set forth in Section 200-2.2 of the Standard Specifications for Public Works Construction. The Class II Aggregate Base should conform to requirements set forth in Section 26-1.02A of the Standard Specifications for California Department of Transportation. Use of other types of base material such as Crushed Miscellaneous Base, Processed Miscellaneous Base, or Milled asphalt concrete is acceptable provided 1 inch is added to the base thickness. Asphalt concrete should be placed in accordance with ‘Standard Specifications for Public Works Construction (Greenbook), Section 302-5. Asphalt concrete pavement should be compacted to at least 95% of Hveem density.

LIMITATIONS

REVIEW, OBSERVATION AND TESTING

The recommendations presented in this report are contingent upon our review of final plans and specifications. Such plans and specifications should be made available to the Geotechnical Engineer and Engineering Geologist so that they may review and verify their compliance with this report and with Appendix J of the California Building Code.

It is recommended that Christian Wheeler Engineering be retained to provide continuous soil engineering services during the earthwork operations. This is to verify compliance with the design concepts, specifications or recommendations and to allow design changes in the event that subsurface conditions differ from those anticipated prior to start of construction.

UNIFORMITY OF CONDITIONS

The recommendations and opinions expressed in this report reflect our best estimate of the project requirements based on an evaluation of the subsurface soil conditions encountered at the subsurface exploration locations and on the assumption that the soil conditions do not deviate appreciably from those encountered. It should be recognized that the performance of the foundations and/or cut and fill slopes may be influenced by undisclosed or unforeseen variations in the soil conditions that may occur in the intermediate and unexplored areas. Any unusual conditions not covered in this report that may be encountered during site development should be brought to the attention of the Geotechnical Engineer so that he may make modifications if necessary.

CHANGE IN SCOPE

This office should be advised of any changes in the project scope or proposed site grading so that we may determine if the recommendations contained herein are appropriate. It should be verified in writing if the recommendations are found to be appropriate for the proposed changes or our recommendations should be modified by a written addendum.

TIME LIMITATIONS

The findings of this report are valid as of this date. Changes in the condition of a property can, however, occur with the passage of time, whether they are due to natural processes or the work of man on this or adjacent properties. In addition, changes in the Standards-of-Practice and/or Government Codes may occur. Due to such changes, the findings of this report may be invalidated wholly or in part by changes beyond our control. Therefore, this report should not be relied upon after a period of two years without a review by us verifying the suitability of the conclusions and recommendations.

PROFESSIONAL STANDARD

In the performance of our professional services, we comply with that level of care and skill ordinarily exercised by members of our profession currently practicing under similar conditions and in the same locality. The client recognizes that subsurface conditions may vary from those encountered at the locations where our borings, surveys, and explorations are made, and that our data, interpretations, and recommendations are based solely on the information obtained by us. We will be responsible for those data, interpretations, and recommendations, but shall not be responsible for the interpretations by others of the information developed. Our services consist of professional consultation and observation only, and no warranty of any kind whatsoever, express or implied, is made or intended in connection with the work performed or to be performed by us, or by our proposal for consulting or other services, or by our furnishing of oral or written reports or findings.

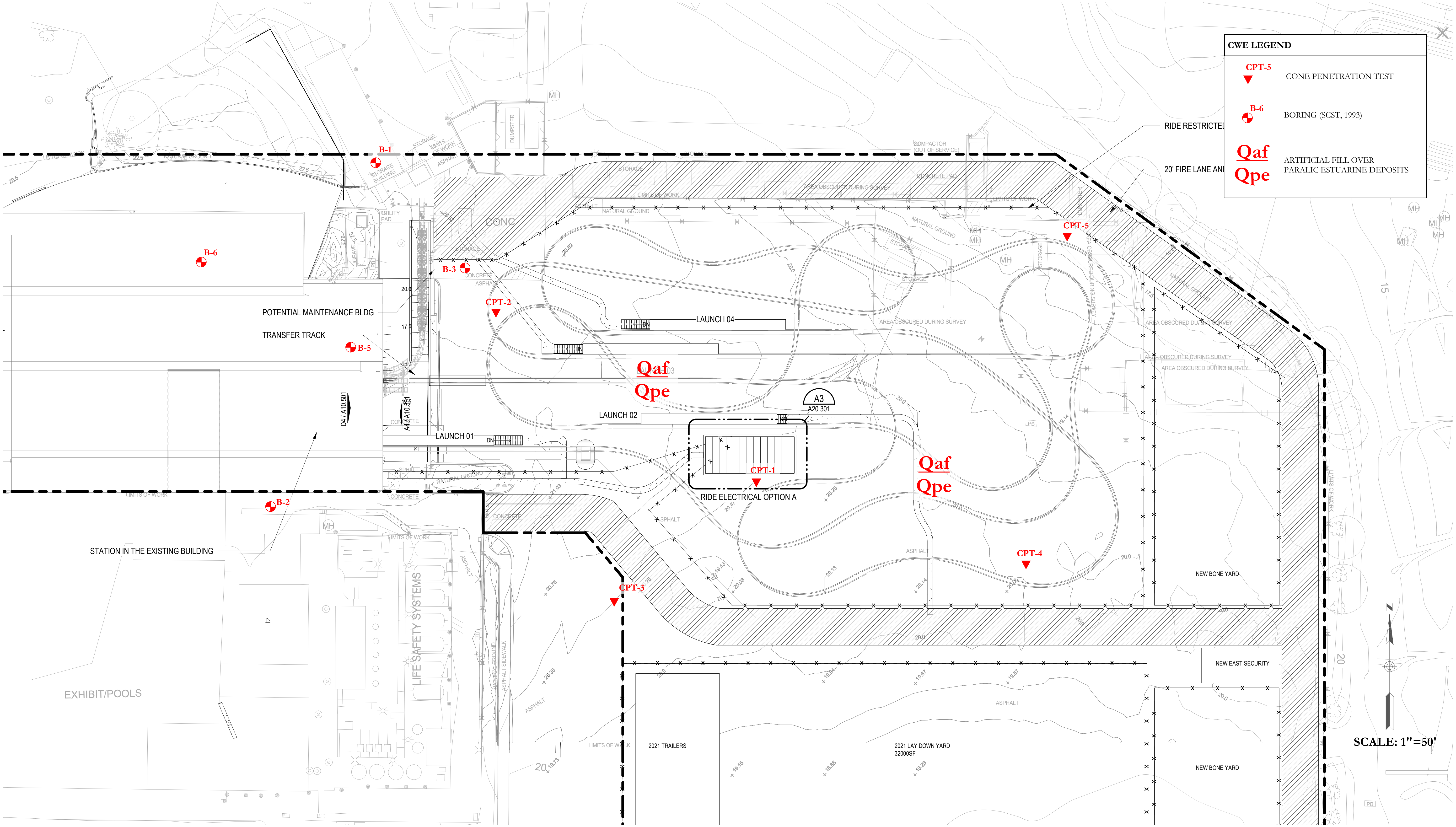
CLIENT'S RESPONSIBILITY

It is the responsibility of SeaWorld, or their representatives to ensure that the information and recommendations contained herein are brought to the attention of the structural engineer and architect for the project and incorporated into the project's plans and specifications. It is further their responsibility to take the necessary measures to insure that the contractor and his subcontractors carry out such recommendations during construction.

FIELD EXPLORATIONS

Five subsurface explorations were made during this investigation at the locations indicated on the Site Plan included herewith as Plate Number 1 on April 2, 2019. These explorations consisted of Cone Penetration Test soundings. The fieldwork was conducted under the observation and direction of our engineering geology personnel.

The CPT probes were performed by Kehoe Testing and Engineering, using an integrated electronic cone system. The results are presented in Appendix A. The CPT soundings were performed in accordance with ASTM Standard D5778. A 30-ton capacity cone was used for all of the soundings. This cone had a tip area equal to 15 square centimeters and friction sleeve area of 225 square centimeters. The cone was designed with an equal end area friction sleeve and a tip end area ratio of 0.85. The fieldwork was conducted under the observation and direction of our engineering geology personnel. On the logs of the CPT soundings, the soils are described in terms of the Soil Behavior Type (SBT). The stratigraphic expression of the soil types, SBT, is based on the relationships between the measured cone bearing, sleeve friction, and penetration pore pressures measured almost continuously within each sounding.



SITE PLAN AND GEOTECHNICAL MAP

SEAWORLD 2021 PROJECT 500 SEAWORLD DRIVE, SAN DIEGO, CA	
DATE: SEPTEMBER 2019	REPORT NO.: 2190160.01
BY: SCC	PLATE NO.: 1

CHRISTIAN WHEELER ENGINEERING