APPENDIX D
Geotechnical Report/Paleontological Resources Assessment

October 27, 2023

EMI Project No. 23-115

Michael Baker International
5 Hutton Centre Drive, Suite 500
Santa Ana, CA 92707

Attention: Mr. Bradley Mielke, PE, SE

Subject: **Draft Foundation Report**
Collins Island Bridge
Newport Beach, California

Dear Mr. Mielke:

Attached is our Draft Foundation Report (DFR) for the proposed replacement of the Collins Island Bridge in the City of Newport Beach, California. This report has been prepared to provide the geotechnical information to the structural designers. This report follows the 2021 California Department of Transportation (Caltrans) Guidelines – Foundation Reports for Bridges, and presents the findings, conclusions and recommendations for the design and construction of the bridge foundations.

Please submit this report to participating agencies for review. All review comments and approved responses will be incorporated into a final report later.

We appreciate the opportunity to provide geotechnical services for this project. If you have any questions, please do not hesitate to contact us.

Sincerely,

EARTH MECHANICS, INC.

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DRAFT FOUNDATION REPORT
COLLINS ISAND BRIDGE
NEWPORT BEACH, CALIFORNIA

Prepared for:

Michael Baker International
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EMI Project No. 23-115

October 27, 2023

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1.0 INTRODUCTION

This Draft Foundation Report presents the findings and conclusions of a geotechnical study conducted by Earth Mechanics, Inc. (EMI) for replacement of the Collins Island Bridge and new seawalls in the City of Newport Beach, California. The purpose of the geotechnical study was to obtain information on subsurface soils and conditions, and develop design and construction recommendations to assist Michael Baker International (MBI) in preparing the project Plans, Specifications, and Estimates (PS&E) for the project.

The geotechnical services provided for this project included the following tasks:

- Collection and review of existing geotechnical information;
- Geotechnical field exploration consisting of exploratory borings and Cone Penetration Test (CPT) soundings;
- Laboratory testing of selected subsurface soil samples;
- Engineering analysis to develop foundation design and construction recommendations; and
- Preparation of this report presenting our findings, conclusions, and recommendations.

2.0 PROJECT DESCRIPTION

The existing Collins Island Bridge provides the only means of vehicle and pedestrian access from Balboa Island via Park Avenue to the residential community on Collins Island (See Figure 1). The existing reinforced concrete bridge was constructed in 1953 and is approximately 20 feet and 8 inches long and 19 feet wide. The bridge is supported on concrete sheet pile bulkheads, which are insufficient to resist current code level seismic loads. The existing bridge accommodates one lane of vehicle traffic, one raised sidewalk, and steel railings on each side of the bridge.

The proposed bridge will be designed to be a total of 20 feet and 6 inches in width to accommodate one vehicle travel lane with 13.75 feet wide, one 4-foot-wide sidewalk, and concrete barriers on each side to provide protection from projected sea level rise. The bridge would be 31 feet in length spanning over existing concrete sheet pile bulkheads. The proposed bridge will be supported on secant piles.

In addition, the seawalls adjacent to the proposed bridge will be improved to accommodate the future water surface to up to El. +14 feet. To minimize the disturbance, the proposed seawall will be installed outside of the existing concrete sheet piles.

All elevations referenced within this report are based on the North American Vertical Datum of 1988 (NAVD 88), unless otherwise noted.

0 4,000 8,000 FEET

REFERENCE: USGS Topographic 7.5 Minute Quadrangle Map - Newport Beach Quadrangle (2018)

Earth Mechanics, Inc. Geotechnical and Earthquake Engineering	COLLINS ISLAND BRIDGE	Site Location Map
	Project No. 23-115 / Date: September 2023	Figure 1

3.0 GEOTECHNICAL INVESTIGATION

EMI conducted a geotechnical field investigation for the proposed bridge replacement consisting of one soil boring and one CPT at Abutment 1 and one CPT at Abutment 2, in May 2023. The locations of the boring and CPTs are shown in Figure 2. Soil exploration information is summarized in Table 1, and LOTB sheet of the recent exploration is included in Appendix A.

TABLE 1. SOIL EXPLORATION INFORMATION

Boring/CPT No.	Boring Type	Approx. Northing	Approx. Easting	Station	Station Line	Offset (feet)	Ground Surface El. (feet)	Bottom of Hole El. (feet)	Groundwater El. During Drilling (feet)
A-23-001	HSA	2169766	6059075	10+81	Park Avenue	9 Rt	+7.8	-63.7	-1.2
CPT-23-001	CPT	2168958	6059290	10+87		4 Rt	+7.7	-90.7	NM
CPT-23-002	CPT	2168930	6059327	11+33		3 Rt	+6.3	-82.6	NM

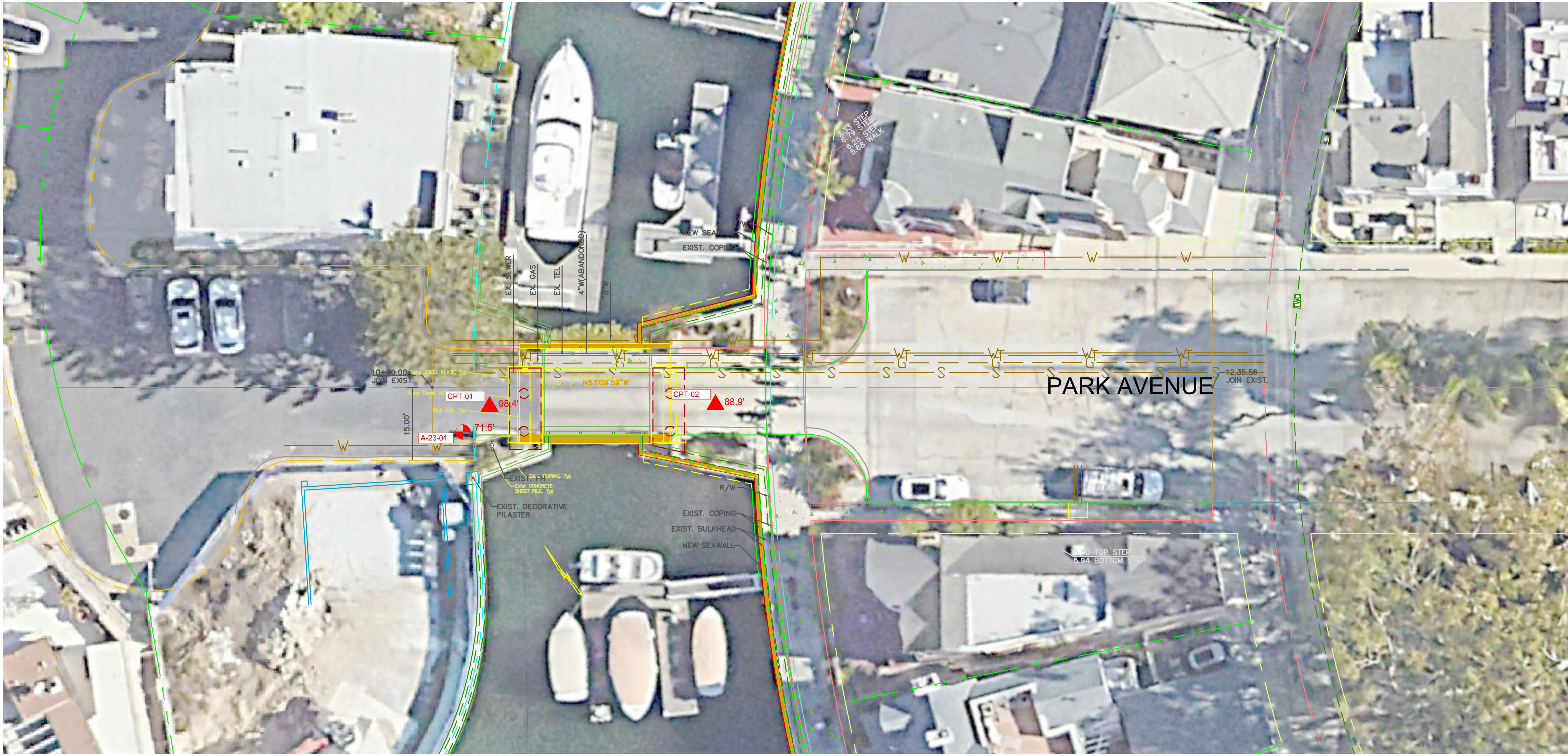
Notes:

(1) Ground Surface Elevations were estimated from topographic plans provided by MBI.

(2) HSA = Hollow Stem Auger, CPT = Cone Penetration Test, NM = Not Measured.

The boring was drilled using a modified CME-75 truck-mounted drill rig equipped with 8-inch diameter hollow-stem augers. Subsurface soils and conditions were logged and samples of soils were collected for laboratory testing. Smaller disturbed and relatively undisturbed soil samples were collected from borings generally at 5-foot intervals using the Standard Penetration Test (SPT) sampler and the Modified California Drive (MCD) sampler, respectively. The SPT sampler is unlined and has an inside diameter of 1.4 inches and the MCD sampler is lined with a series of 1-inch tall brass rings with an inside diameter of 2.4 inches.

Blowcounts from the SPT and MCD samplers were recorded during the exploration. The samplers were driven using a 140-lb hammer falling 30 inches down a total depth of 18 inches or until refusal, whichever occurs first. An automatic trip hammer was used by the drilling contractor, and this hammer had a rated efficiency of 88% (hammer efficiency provided by the drilling contractor). The blowcounts for the last 12 inches or less of penetration were recorded and are shown in the LOTB sheet included in Appendix A.



A-23-01
Depth (ft) Location of Geotechnical Boring

CPT-01
Depth (ft) Location of Cone Penetrometer Test Sounding

0 20 40 FEET
SCALE

Figure 2

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

Collins Island Bridge
Boring Location Plan

The CPT soundings were performed using an electronic cone penetrometer in general accordance with current ASTM Standards (ASTM D5778 and ASTM D3441). The CPT equipment consisted of a cone penetrometer assembly mounted at the end of a series of hollow sounding rods. The cone penetrometer assembly consisted of a conical tip with a 60˚ apex angle and a projected cross-sectional area of 2.33 in^2 (15 cm^2) and a cylindrical friction sleeve with a surface area of 23.25 in^2 (150 cm^2). The interior of the cone penetrometer is instrumented with strain gauges that allow simultaneous measurements of cone tip and friction sleeve resistance during penetration. The cone penetrometer assembly is continuously pushed into the soil by a set of hydraulic rams at a standard rate of 0.79 inch per second (20 mm per second) while the cone tip resistance and sleeve friction resistance are recorded every 1.967 inches (50 mm) and stored in digital form. A specially designed all-wheel drive 30-ton truck provides the required reaction weight for pushing the cone assembly and is also used to transport and house the testing equipment. The computer-generated graphical logs include tip resistance, friction resistance, and friction ratio. Soil behavior type interpretations are based on guidelines by Robertson (2009). Seismic Cone Penetration Test (SCPT) was also used for the soundings to obtain in-situ shear wave velocity. The shear wave is generated using an air-actuated hammer, which is located inside the front jack of the CPT rig. The cone has a triaxial geophone, which recorded the shear wave signal generated by the air hammer.

4.0 LABORATORY TESTING PROGRAM

Selected soil samples were tested to determine soil classification and physical and engineering properties. A list of tests performed, the corresponding test methods, and purpose of testing is presented in Table 2.

The laboratory soil tests were conducted in general accordance with California Test (CT) methods or American Society for Testing and Materials (ASTM) standards. The test results are presented in Appendix B. The locations where tests were performed are shown on the LOTB sheets included in Appendix A.

TABLE 2. EXPLANATION OF LABORATORY TESTS PERFORMED

Type of Test	Applicable Test Method	Purpose
Dry Density	ASTM D 2937	Estimate in-situ soil density
Moisture Content	ASTM D 2216	Estimate in-situ soil moisture content
No. 200 Wash	ASTM D 1140	Estimate percentage of gravel, sand, and fines content
Atterberg Limits	ASTM D 4318	Evaluate plasticity of fine-grained particles
UU Triaxial Test	ASTM D 2850	Determine stress-strain relationship of cohesive soil
Direct Shear	ASTM D 3080	Estimate strength parameters
Soil pH	CT 532/643	Determine corrosion potential
Minimum Resistivity	CT 532/643	
Sulfate Content	CT 417	
Chloride Content	CT 422	

5.0 GEOTECHNICAL CONDITIONS

5.1 Geology

5.1.1 Physiography

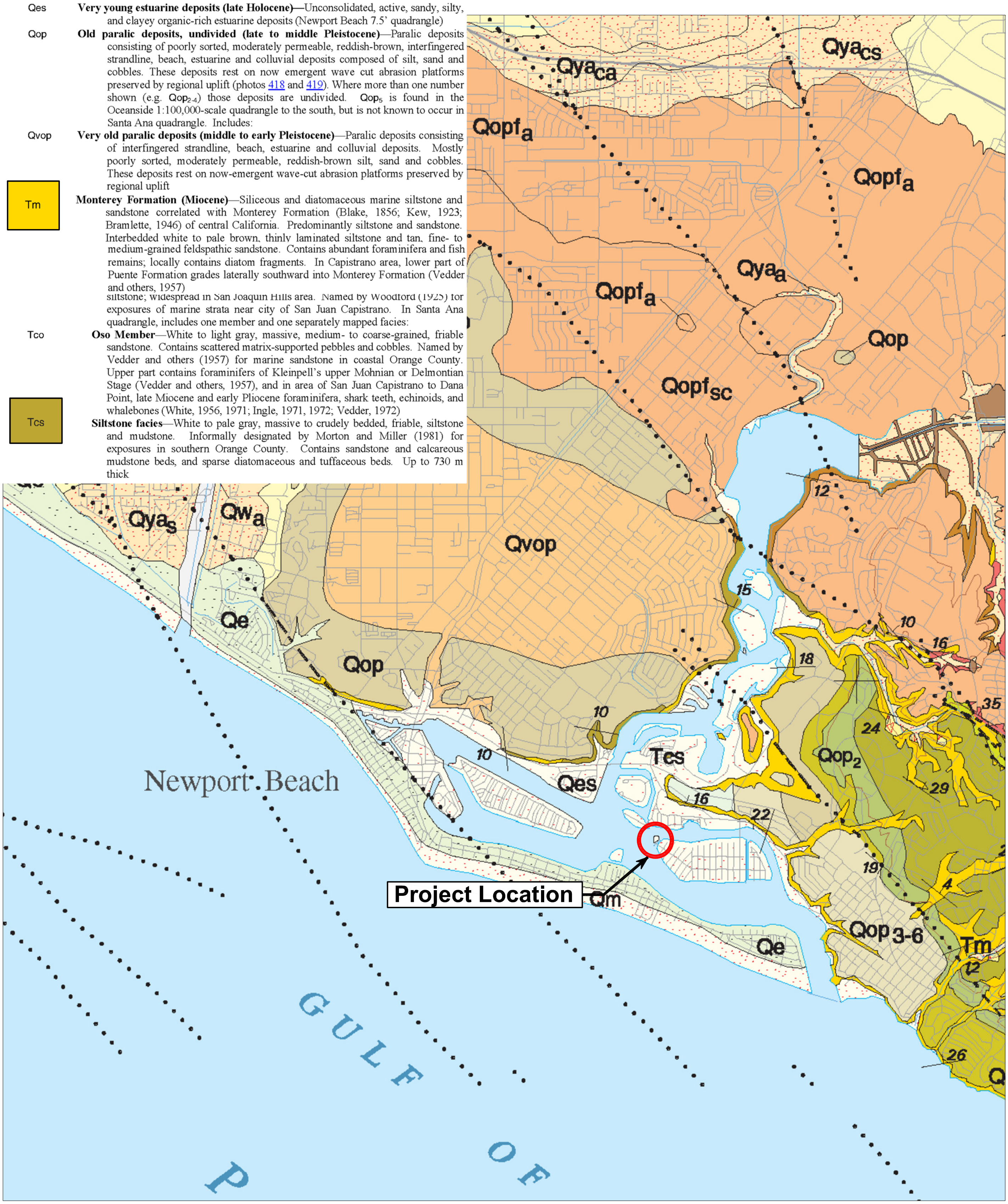
The project site is in Southern California within the Peninsular Ranges physiographic province in the Orange County Coastal Plain. The Orange County Coastal Plain is a broad, gently dipping alluvial plain that extends from the Santa Ana Mountains to the Pacific Ocean. The project site is at the far south-western edge of the Coastal Plain, in the Newport Bay. The Orange County Coastal Plain has been created by run-off from the Santa Ana Mountains covering the area with layers of sediment. The site is generally underlain by hydraulic fill, which was used originally to create the island. Underlying the hydraulic fill are alluvial soils deposited into the bay by way of the Santa Ana River (before being re-aligned). These deposits generally consist of grey, fine sands and silts. Underlying the alluvial deposits is the sedimentary bedrock composed of dark to medium brown, well consolidated, highly fractured fine siltstone and claystone of the Capistrano Formation. See Figure 3 for Regional Geologic Map.

The geologic formations in the area, following the nomenclature of Morton and Miller (2006) in descending stratigraphic order are:

- Hydraulic Fill, Holocene (Af);
- Alluvial Deposits, Holocene (Qa); and
- Capistrano Formation, Pleistocene (Tu1)
- Monterey Formation, Miocene (Tm).

5.1.2 Geologic Structure

The geologic structure at the site is characterized by relatively flat-lying Quaternary-age strata underlain by ancient basement rocks and the result of the Newport-Inglewood Structural Zone. Newport Mesa is a large uplifted geomorphic feature created by faulting along the NISZ that is adjacent to Newport Bay. Geologic structure within the site vicinity consists of deformed, faulted, and folded bedding associated with the NISZ, with regional onshore data showing beds dipping shallowly to the north and west between 15 and 25 degrees. The most influential faults in the vicinity include the Newport-Inglewood Structural Zone, THUMS-Huntington Beach fault, Pelican Hill fault and San Joaquin Hills thrust fault.



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REFERENCE: Morton, P.K., and Miller, R.V., 2006, Geologic map of the San Bernardino and Santa Ana 30' x 60' quadrangles, California. U.S. Geological Survey: Open-File Report 2006-1217.

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COLLINS ISLAND BRIDGE				Regional Geologic Map
Project No.	23-115	Date:	October 2023	Figure 3

5.1.3 Faulting

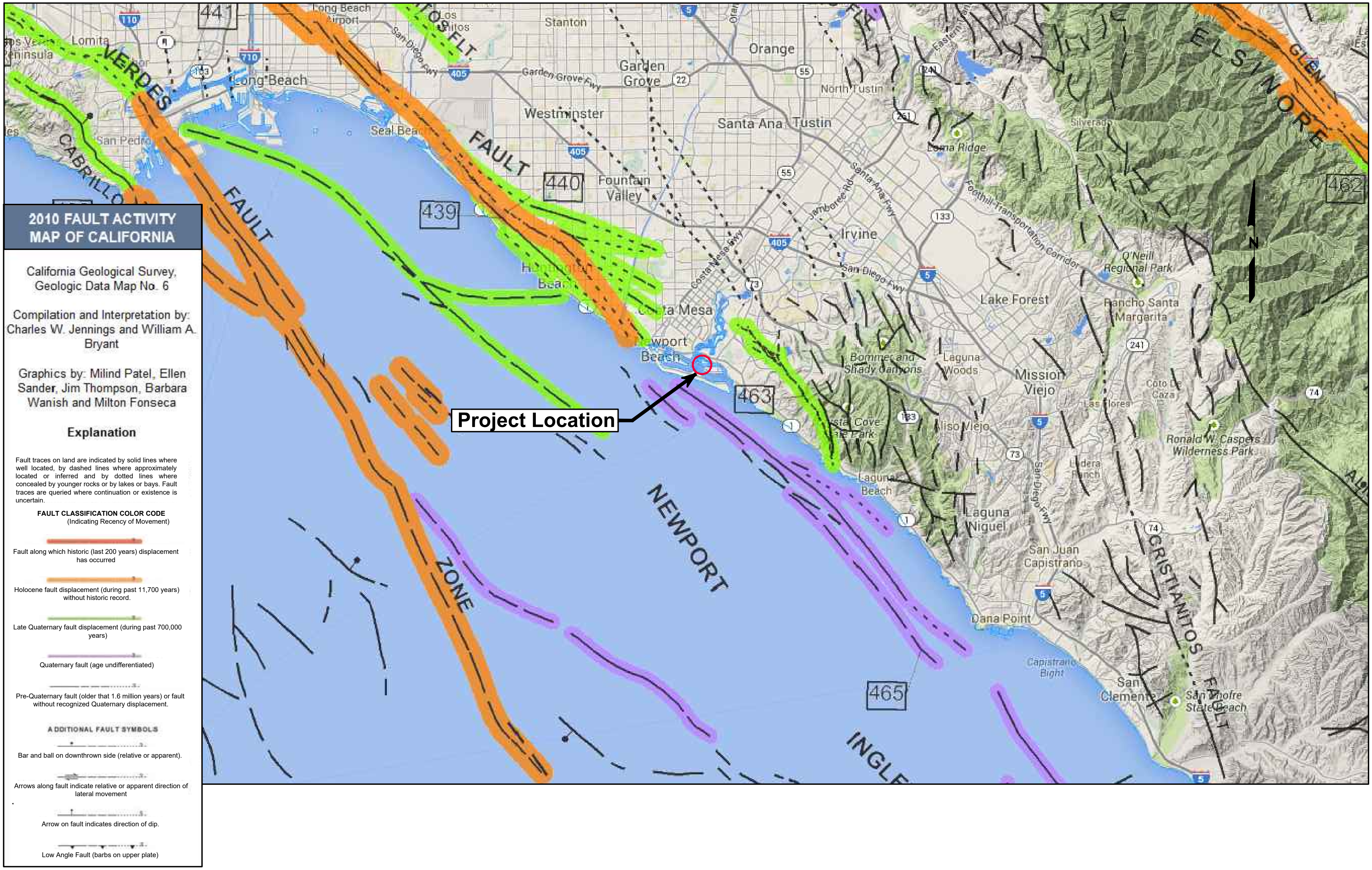
The project site is located within the Salinas Basin region of the Coast Ranges Geomorphic Province. The region consists of numerous active and potentially active faults including the Newport-Inglewood Structural Zone, the Pelican Hill fault and the San Joaquin Hills fault. Of these faults, the NISZ is the nearest fault identified as Alquist-Priolo (AP) Earthquake Fault Zones defined by the Alquist-Priolo Earthquake Hazards Act of 1972 revised in 1994. The AP faults not only represent earthquake shaking hazards but have a potential for surface ground rupture. The type and magnitude of the seismic hazard affecting the site are dependent on the distance and causative faults and the intensity and magnitude of the seismic event. Other potentially active faults may not be identified as AP Earthquake Fault Zones because their locations are not well defined and/or they have not generated earthquakes in historical time. Locally, smaller faults exist within the valley floor within the vicinity of the site location as well. The project site does not enter into any AP fault zones and does not cross any active fault traces (Figure 4).

Newport-Inglewood Structural Zone. The Newport-Inglewood Structural Zone (NISZ) is a northwest to southeast trending fault system and is considered active by the State of California and an Alquist-Priolo Earthquake Fault Zone has been established around the fault (CGS, 1986). The NISZ comprises a zone of faults and folds transecting the western Los Angeles Basin and is 56 miles long extending onshore from the Santa Monica Mountains to the San Joaquin Hills and Newport Bay area, and then continues offshore to approximately the Dana Point area. The overall fault system is generally right-lateral strike slip and it is understood to be capable of generating a magnitude of up to 7.4 (Mw) (Grant, Shearer, 2004). In north San Diego County, and south of where the NISZ disappears in the Dana Point area, the Rose Canyon Fault continues directly south and along the same alignment as the NISZ. It is believed that these two fault systems may actually be one fault system; however, more research is needed to determine the relationship (Grant, Shearer, 2004).

The NISZ has had numerous earthquakes occur within recent time including the Long Beach earthquake in 1933, Inglewood in 1920, Gardena in 1941, and Torrance-Gardena in 1941. The project site is located approximately 2.6 miles southeast of the nearest mapped trace of the NISZ.

THUMS-Huntington Beach Fault. The THUMS-Huntington Beach fault is a continuous right-slip fault zone with three segments and two steps that extends southeastward from the Huntington Beach anticline and merges with the Newport Inglewood fault zone. The fault branches from the Palos Verdes fault zone to form the southwestern border of the Wilmington and Huntington Beach anticlines. The fault should be considered active as it is closely related to the Palos Verdes and Newport-Inglewood fault zones with a possible transfer of slip to or from both fault systems. The fault is located approximately 4.6 miles southwest of the project area.

San Joaquin Hills Thrust Fault. The San Joaquin Hills fault is a blind thrust fault located northeast of the project site beneath the San Joaquin Hills. The project site is located approximately 5.8 miles south of the projected trace according to USGS. The recent uplift of the San Joaquin Hills has been interpreted to be the result of slip along the San Joaquin Hills blind thrust fault.



Earth Mechanics, Inc. Geotechnical and Earthquake Engineering	COLLINS ISLAND BRIDGE	Regional Fault Map
	Project No. 23-115 / Date: October 2023	Figure 4

Research by Grant et al. (1999, 2002) on the age and rate of uplift of the San Joaquin Hills included a postulation that uplift of the hills was due to the presence of the buried, low-angle, blind, thrust fault below the San Joaquin Hills and, furthermore, that the fault is capable of generating magnitude 6.8 to 7.3 earthquakes. They postulate that the fault dips westerly from about one mile deep under the east side of the San Joaquin Hills to about five miles deep where the fault would intersect the Newport-Inglewood fault. They did not discuss the difficult and critical issue of how the two faults interact where they intersect.

There is no direct evidence for a subsurface thrust fault under the San Joaquin Hills. For example, there are no boreholes showing a fault, no geophysical evidence (seismic reflection or refraction), and no seismological evidence indicating such a fault. A recent small earthquake in the area was predominantly a strike-slip rupture of the type expected on the NISZ rather than a thrust-type event that one would expect from the postulated subsurface fault.

As visualized by Grant et al (2002), the fault would dip southwesterly such that it would not directly underlie the site but at its closest point to the site would be about five miles laterally in the subsurface. The fault is not recognized as an active fault according to the Alquist-Priolo Earthquake Fault zone maps.

Pelican Hill Fault. The Pelican Hill fault is a right-lateral strike slip fault that is located approximately 2.1 miles northeast of the project site. The fault is considered potentially active though its latest activity is believed to have occurred between the early Miocene and late Pliocene.

5.2 Geological Hazards

The proposed bridge site is located off Collins Island located in the Newport Bay. Elevations at the abutments of the bridge approaches range between +7 and +8 feet.

Liquefaction and Lateral Spreading. According to the Seismic Hazard Map (CGS, 1997-1998), (Figure 5) the near-surface alluvial sediments within the project area are susceptible to liquefaction due to moderate to intense ground shaking. Further analysis and potential for liquefaction is discussed in more detail in Section 10.2.

Fault Rupture/Seismic Shaking. There are no known active surface faults within the project limits, so the potential for ground rupture is considered low. The nearest active or potentially active fault is located approximately 2 to 3 miles from the project site. As a result, moderate to intense ground shaking should be anticipated within the project area in the event of an earthquake. Additional discussion of ground rupture is included in Section 10.3.

Slope Stability. No natural slopes are present within or in the vicinity of the site. So, landsliding of natural existing slopes is not a design issue. Existing shoreline slopes are presumed to be constructed of properly and protected with rip rap and should be considered to be grossly stable.

0 4,000 8,000 FEET

SCALE 1" = 4,000'

REFERENCE:
California Geological Survey, 1997 and 1998, Seismic Hazard Zones, Newport Beach Quadrangle California Division of Mines and Geology, Official Map, April 17 and April 15.

Earth Mechanics, Inc. Geotechnical and Earthquake Engineering	**COLLINS ISLAND BRIDGE**	Seismic Hazard Map
	Project No. 23-115 / Date: September 2023	Figure 5

Expansive Soils. Expansive soils swell or heave with increases in moisture content and shrink with decreases in moisture content. Montmorillonitic clays are most susceptible to expansion due to their layered crystalline structure. Claystone beds within Capistrano Formation may have potential to be highly plastic and expansive. As part of the laboratory testing program, plasticity index testing will be conducted on any clayey soils/rock encountered during the site-specific geotechnical field investigation.

Tsunami/Flooding. Tsunamis, or seismic sea waves, are large oceanic waves generated by earthquakes, submarine volcanic eruptions or large submarine landslides. They are capable of traveling long distances across ocean basins, and can force large quantities of water up onto shore at high velocities. The forces involved with tsunamis are of such large magnitude that the only positive means of protection is to avoid areas subject to tsunamis. The largest tsunami reported in California followed the 1812 earthquake, in which sea waves as large as 30 to 35 feet reached Santa Barbara. The project site is located within a tsunami inundation according to published inundation maps (CGS, 2009). The probability and severity of tsunami inundation cannot be estimated based on current available information.

5.3 Subsurface Conditions

The subsurface information indicates that the site soils are composed predominantly of coarse-grained soils consisting of loose to medium dense sand at the upper 20 feet. Below that is an approximately 30 feet thick of dense to very dense sand over the sedimentary bedrock (siltstone to claystone).

The idealized soil/rock profile and design strength parameters for geotechnical analyses and foundation design are presented in Table 3. In Table 3, a factor of 0.65 was used to convert Modified California Drive (MCD) sampler blowcounts to Standard Penetration Test (SPT) sampler blowcounts. The equivalent SPT N_{60} blowcounts were obtained from CPT soundings following Robertson (2012) procedures. The shear strength parameters were estimated using laboratory test data and correlations with field blowcounts (Lam and Martin, 1986). In locations where a discrepancy was observed between blowcount correlations and laboratory test results, the design strength parameters were selected using the blowcount correlations considering that the blowcount correlations provide the best indication of in-situ soil strength.

It should be noted that the idealized soil profiles and shear strength parameters in Table 3 were developed primarily for the design of bridge foundations addressed in this report. Direct application of the same idealized profiles and shear strength parameters for other design elements not specifically addressed in details in this report are likely to be invalid. This is because selecting an idealized profile and shear strength parameters, to some extent, is influenced by the preferred design methodologies associated with bridge foundations. The same is true for the laboratory test results: the type and distribution of testing were tailored to bridge foundation design. Selective usage of one or multiple sets of test results for other design elements not specifically addressed in detail in this report will likely provide an erroneous interpretation of onsite soil properties. For design elements not specifically addressed herein, we recommend supplemental field exploration and laboratory tests be performed to establish suitable and representative geotechnical design data for the specific design element.

TABLE 3. IDEALIZED SOIL/ROCK PROFILE AND STRENGTH PARAMETERS

Approximate Elevation (feet)	Predominant Soil/Rock Type	Range of Corrected SPT N_{60} Blowcount (Blows/ft)	Friction Angle (degrees)	Cohesion or Undrained Shear Strength (psf)	Total Unit Weight (pcf)
+8 to -12*	Loose to Medium Dense Sand and Sand with Silt*	7 to 22 Average = 12	30	75	115
-12 to -42	Dense to Very Dense Sand	30 to >50 Average = 40	37	50	125
-42 to -90	Sedimentary Bedrock: Siltstone/ Claystone	15 to >50 Average = 30	-	4,000	125

*: this layer is potentially liquefiable. The undrained shear strength of 200 psf for liquefiable soil was estimated per the procedure listed in Caltrans Memo to Designer 20-15 (2017) under seismic condition.

6.0 GROUNDWATER

Based on recent field investigation, the shallowest groundwater was encountered at El. -1.2 feet in A-23-001. However, the high tide water elevation is at El. +7.7 feet as shown on the general plan. Therefore, a groundwater elevation of +7.7 feet is used for the soil liquefaction evaluation and foundation design.

7.0 AS-BUILT DATA

Based on a review of the as-built plans, the existing foundation information is summarized in Note that the elevations listed in the table below were based on the elevations listed in as-built plans. For as-built plans earlier than 1989, the elevations were based on the National Geodetic Vertical Datum of 1929 (NAVD 29).

Table 4. Note that the elevations listed in the table below were based on the elevations listed in as-built plans. For as-built plans earlier than 1989, the elevations were based on the National Geodetic Vertical Datum of 1929 (NAVD 29).

TABLE 4. SUMMARY OF AS-BUILT FOUNDATION DATA

Support	Foundation Type	Design Load (kips)	Approximate Pile Cap Bottom Elevation (feet)	Approximate As-Built Pile Tip Elevation (feet)
Abutment 1	10”x3’x18’ Precast Sheet Piles	Unknown	+5.97	-12.0
Abutment 2			+6.17	-11.8

8.0 SCOUR AND DREDGING

Based on our discussions with the structural designers, scour is not an issue for the subject bridge but dredging operation is ongoing. Unfortunately, the dredging depth and subsequent cycles of dredging and re-deposition of sediments in the dredged zone are all unknown. The mudline shown in the general plans is about El. -4 feet. Therefore, a lowering of the mudline of 5 feet (i.e El. -9 feet) will be considered for the Service I and Strength Limit State load cases and no lowering of the mudline was considered for the Extreme Event Limit State load case (i.e El. -4 feet).

9.0 CORROSION EVALUATION

Representative soil samples were tested to determine corrosivity including minimum resistivity, pH, soluble sulfate content, and soluble chloride content. Two soil samples were tested for corrosivity and the results are summarized in Table 5.

TABLE 5. SOIL CORROSION TEST RESULTS

Boring No.	Sample No.	Sample Depth (feet)	USCS Soil Type	Minimum Resistivity (ohm-cm)	pH	Chloride Content (ppm)	Sulfate Content (ppm)
A-23-001	S-1	5	Poorly Graded Sand with Silt (SP-SM)	140	8.3	1,030	1,680
	D-11	60	Organic Silt (OH)	91	7.3	565	4,068

According to the Caltrans Corrosion Guidelines V3.0 (Caltrans, 2021b), soils are considered corrosive if the pH is 5.5 or less, or chloride concentration is 500 parts per million (ppm) or greater, or sulfate concentration is 1,500 ppm or greater. Based on the above corrosion test results and the Caltrans criteria, the on-site soil samples are considered to be corrosive.

Therefore, a minimum concrete cover should be in accordance with Table 5.10.1-1 of the California Amendments to the AASHTO LRFD Bridge Design Specifications (Caltrans, 2019a) for chloride concentration more than 500 ppm. Corrosion resistant concrete mix designs that address corrosive conditions are specified in Section 90-1.02H of the Caltrans Standard Specifications (2023b).

10.0 SEISMIC INFORMATION

10.1 Ground Motion

The design ARS curves were determined using the Caltrans ARS Online V3.1.0 (2023a), following the procedures described in Caltrans Seismic Design Criteria Version 2.0 (SDC 2.0) (2019c) and October 2019 Interim Revisions to SDC 2.0 (2019b), and the small-strain shear wave velocity for the upper 100 feet (V_{s30}). This V_{s30} was estimated from the SCPT in-situ shear wave velocity measurements and from the information presented in the LOTB sheet included in Appendix A and the SPT correlations provided in the Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendations (Caltrans, 2012). The key parameters for determining the design ARS curves are listed in Table 6.

TABLE 6. KEY PARAMETERS FOR DETERMINING DESIGN ARS CURVE

Site Coordinates	Latitude = 33.6083 degrees	Longitude = -117.9000 degrees
Shear Wave Velocity, V_{s30}	934 feet/sec (285 m/sec)	

The design ARS curves are presented in Figure 6. The design magnitude (M) is 6.59 and the mean site-to-fault distance at 1.0 second period is 14.2 miles. The Peak Ground Acceleration (PGA) is 0.49g. Based on the subsurface information collected from the LOTB sheet included in Appendix A and per Sections 6.1.2 and 6.1.3 of SDC 2.0 (2019c), the onsite soils are classified as “Class S2” soils.

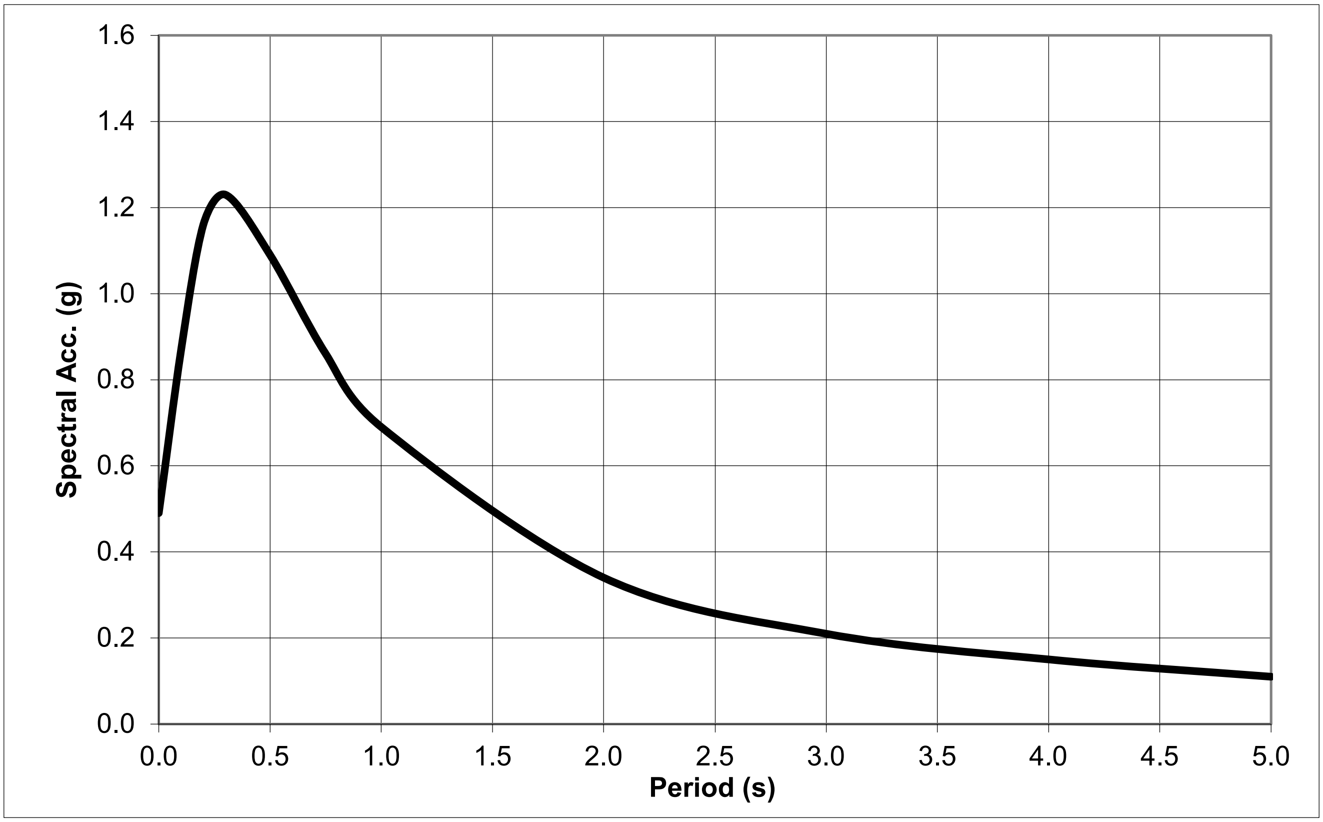
10.2 Liquefaction Potential and Seismically-Induced Settlement

The liquefaction potential screening followed the Caltrans Geotechnical Manual – Liquefaction Evaluation (2020), which used the liquefaction procedure by Youd et al. (2001) with the analysis depth of 70 feet and a factor of safety against liquefaction of 1.0. Based on a design groundwater elevation of +7.7 feet and two site-specific CPTs, results of the analyses indicate that granular materials susceptible to liquefaction were encountered. These potentially liquefiable soil layers are located between El. +8 and -12 feet for both CPTs.

In addition to the reduction in soil strength, liquefaction will also result in seismically-induced settlements. In the liquefiable layers, seismically-induced soil settlements are expected to be up to 4.5 inch. These settlements will generate downdrag forces on the piles, which will be considered in foundation design.

10.3 Ground Rupture

No major faults traverse through the project site. The California Division of Mines and Geology has not identified Alquist-Priolo Fault Zones through the site. Therefore, the risk of ground surface rupture and related hazards at the project site are expected to be low. According to Caltrans Memo To Designers 20-10 (Caltrans, 2013), since the project site does not fall within an Alquist-Priolo Earthquakes Fault Zone or within 1,000 feet of an unzoned fault that is Holocene or younger in age, further fault studies will not be needed.



Design ARS Curve

Latitude: 33.6083 Longitude: -117.9000

Shear Wave Velocity (V_{s30}) = 285 m/s

Design Magnitude (M) = 6.59

Peak Ground Acceleration (PGA) = 0.49g

5% Damping

Spectral Coordinates:

Period (sec)	**Acc.** (g)	**Period** (sec)	**Acc.** (g)
0.00	0.49	1.00	0.69
0.10	0.87	2.00	0.34
0.20	1.16	3.00	0.21
0.30	1.23	4.00	0.15
0.50	1.09	5.00	0.11
0.75	0.86		

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

Project Number : 23-115 | Date: June 2023

Collins Island Bridge

Design Acceleration Response Spectrum

Figure 6

11.0 GEOTECHNICAL RECOMMENDATIONS

11.1 Bridge Design

11.1.1 Foundation Type

Per Section 10.2, the upper 20 feet of soils is considered liquefiable so spread footing is not suitable for the project site. Pile foundation is considered feasible. Since the seawall in front of the proposed bridge will not be retrofitted or replaced and the wall condition is also in question, secant pile wall abutments are proposed, which is a series of alternating reinforced cast-in-drilled-hole (CIDH) piles and un-reinforced concrete piles. It can serve as both functions of the bridge foundation and seawall, which is similar to the existing bridge.

In addition, to maintain the traffic to Collins Island, a stage construction is proposed to keep a traffic lane open during construction.

11.1.2 Axial Pile Capacity

Load Resistance Factor Design (LRFD) is used for foundation design. The foundation design data sheet and foundation factored design loads were provided by the bridge designers following the latest Caltrans Memo to Designers 3-1 (Caltrans, 2014), and are shown in Table 7 and Table 8, respectively. Note that the axial design is only for the reinforced CIDH piles.

TABLE 7. FOUNDATION DESIGN DATA SHEET

Support No.	Pile Type	Finished Grade Elevation (feet)	Cut-off Elevation (feet)	Pile Cap Size (feet)		Permissible Settlement under Service Load (inch)	Number of Piles per Support
				B	L		
Abut 1	24" CIDH	~ +7.4	+5.9	3	25	1	3
Abut 2	24" CIDH	~ +7.1	+5.6	3	25	1	3

TABLE 8. FOUNDATION FACTORED DESIGN LOADS

Support No.	Service-I Limit State (kips)		Strength/Construction Limit State (Controlling Group, kips)				Extreme Event Limit State (Controlling Group, kips)			
			Compression		Tension		Compression		Tension	
	Total Load Per Support	Perm. Loads Per Support	Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile
Abut 1	180	109	262	87	0	0	109	36	0	0
Abut 2	180	109	262	87	0	0	109	36	0	0

The axial capacities of CIDH piles were estimated using the computer program SHAFT v2017 (Ensoft, 2017). The axial pile capacities are based on soil resistance only and may be further limited by the pile-head connection details and structural material strength. The skin frictions obtained from Shaft results were reduced using a factor of 0.63 (=2/pi) to consider the efficiency of closely spaced adjacent piles. Only skin friction was included in the axial capacities and end bearing was ignored. The calculated pile tip elevations are presented in Table 9. The pile data table is presented in Table 10. As mentioned above, the pile data table is for the reinforced CIDH piles. The unreinforced concrete piles should be tipping to El. -17 feet, which is 5 feet below the competent materials as shown in. Table 3, for the slope stability purpose.

Since the secant pile abutments are also designed as a backup seawall in case the existing wall is not functioning, the axial capacity above the mudline discussed in Section 8 (El. -9 feet) is ignored for the service and strength limit states. For the extreme case, the downdrag force of 32 kips between the cutoff elevation and the bottom of the liquefiable soils (El. -12 feet) were added to the pile load assuming that the existing seawall is still intact (i.e downdrag force induced from both sides of piles) for the worst scenario.

TABLE 9. FOUNDATION DESIGN RECOMMENDATIONS

<table>
<tr><th rowspan="3">Sup. No.</th><th rowspan="3">Pile Type</th><th rowspan="3">Cut-off El. (feet)</th><th rowspan="3">Service-I Limit State Load per Support (kips)</th><th rowspan="3">Total Permissible Settlement (inches)</th><th colspan="4">Nominal Resistance (kips)</th><th rowspan="3">Design Tip El. (feet)</th><th rowspan="3">Spec. Tip El. (feet)</th></tr>
<tr><th colspan="2">Strength / Construction</th><th colspan="2">Extreme Event</th></tr>
<tr><th>Comp φ=.7</th><th>Tens φ=.7</th><th>Comp φ=1</th><th>Tens φ=1</th></tr>
<tr><td>Abut 1</td><td>24" CIDH</td><td>+5.9</td><td>180</td><td>1</td><td>130</td><td>0</td><td>40</td><td>0</td><td>-48(a-I)
-32(a-II)
-19(c)</td><td>-48</td></tr>
<tr><td>Abut 2</td><td>24" CIDH</td><td>+5.6</td><td>180</td><td>1</td><td>130</td><td>0</td><td>40</td><td>0</td><td>-48(a-I)
-32(a-II)
-19(c)</td><td>-48</td></tr>
</table>

Notes:

(1) Design tip elevations are controlled by: (a-I) Compression (Strength Limit), (a-II) Compression (Extreme Limit), (c) Settlement, (d) Lateral Load.

(2) The Specified Tip Elevation shall not be raised.

(3) Column heading modified from “Required Factored Nominal Resistance” to “Nominal Resistance”.

(4) The lateral pile tip will be determined by the structural designers.

TABLE 10. PILE DATA TABLE

Support No.	Pile Type	Nominal Resistance (kips)		Design Tip Elevation (feet)	Specified Tip Elevation (feet)
		Compression	Tension		
Abut 1	24" CIDH	130	N/A	-48(a) -19(c)	-48
Abut 2	24" CIDH	130	N/A	-48(a) -19(c)	-48

Notes:

(1) Design Tip Elevations are controlled by the following demands: (a) Compression, (b) Tension, (c) Settlement, and (d) Lateral Loads.

(2) The Specified Tip Elevation shall not be raised.

(3) The lateral pile tip will be determined by the structural designers.

11.1.3 Lateral Pile Solutions

Nonlinear soil resistance (p) versus pile deflection (y) curves estimated using the computer program LPILE (Ensoft, 2019a) based on Table 11 were provided to the structural designers to estimate the lateral pile capacity. Both liquefaction and scour (or dredge) are considered in the model. As discussed earlier, the secant pile wall abutment will be designed in case of no existing seawalls in front.

The pile spacing of the reinforced CIDH piles is assumed to be 42 inches, which is based on 3-inch overlapping with 24-inch diameter unreinforced concrete piles. With the assumed pile spacing, a p-multiplier of 0.6 can be used along the longitudinal direction based on the Ensoft Pywall Technical Manual (Ensoft, 2019b). For the liquefied soils, a p-multiplier of 0.07 can be used following the procedure listed in Caltrans Memo to Designer 20-15 (2017) under the seismic conditions.

TABLE 11. LPILE INPUT PARAMETERS

p-y Curve Model	Top of Layer Elevation (feet)	Bottom of Layer Elevation (feet)	Effective Unit Weight (pcf)	Friction Angle (Deg.)	Undrained Shear Strength (psf)	k (pci)	ε_{50}
Abutments (Strength Limit)							
API Sand (O'Neill)	-9	-12	52.6	30	0	35	-
API Sand (O'Neill)	-12	-42	62.6	37	0	110	-
Stiff Clay w/o Free Water (Reese)	-42	-90	62.6	0	4,000	-	0.005
Abutments (Extreme Limit)							
API Sand (O'Neill)[2]	-4	-12	52.6	30	0	35	-
API Sand (O'Neill)	-12	-42	62.6	37	0	110	-
Stiff Clay w/o Free Water (Reese)	-42	-90	62.6	0	4,000	-	0.005

Notes:
(1) A p-multiplier of 0.6 can be used for the LPile analysis along longitudinal direciton
(2) This layer is liquefiable. a p-multiplier of 0.07 should be used for seismic conditions.

11.1.4 Bridge Abutment Wall Earth Pressures

If walls are free to move laterally at the top, a static active lateral earth pressure of 20 psf per foot of depth is recommended in addition to a hydrostatic pressure of 62.4 pcf for the portion of abutment wall above the design mudline described in Section 8.

A uniform lateral pressure due to traffic loading, equivalent to a vertical pressure produced by at least 2 feet of earth with a soil unit weight of 120 pcf, should be added to the above lateral earth pressure. Therefore, for abutment walls that are free to move laterally at the top, the recommended uniform lateral earth pressure is 80 psf.

The seismic earth pressures were estimated following Caltrans design criteria using one third of Caltrans PGA of 0.49g. The seismic incremental earth pressure should be modeled as a triangle with an equivalent fluid pressure of 35 psf per foot of depth, which is a larger of the seismic earth pressures due to 30 degree of sand materials and due to 200 psf of undrained shear strength of liquefiable soil.

11.1.5 Approach Embankments

Settlement and Settlement Period. Based on the profiles provided by the designers, the finish grade and existing grade of the approaches at both abutments are similar. Therefore, we don't expect any embankment settlement. The settlement period is not required.

Global Stability. Global stability analyses were conducted for both static and pseudo-static conditions for potential deep-seated failures below the abutment footing. The analysis was performed using the computer program Slide2 (Rocscience, 2020).

Slope stability analyses were conducted for the static condition including a 2-foot soil surcharge to represent traffic loading. In accordance with Caltrans guidelines (2014a), stability analysis for the seismic condition was performed using the pseudo-static approach with a seismic coefficient of 0.163, which is equal to one-third PGA.

Under the seismic conditions, both cases of 30 degree of sand materials and 200 psf of liquefiable soil were checked for the liquefiable layer (upper 20 feet).

According to the results of the analyses, the proposed models meet the minimum required factor-of-safety for deep-seated failure of 1.5 for the static condition and 1.1 for the pseudo-static condition per Caltrans guidelines (2014a).

11.2 Design of Seawalls

The proposed seawalls are located in front of the existing seawalls. Design of seawalls is assumed in case of no existing seawalls. The seawalls are proposed using either sheet piles or king piles with sheet piles. At the time of preparing this report, the pile type is still under evaluation.

Please note that the sheet piles should be embedded at least 5 feet below the competent soils, which is similar to the bridge unreinforced piles, if the king piles with sheet piles are used.

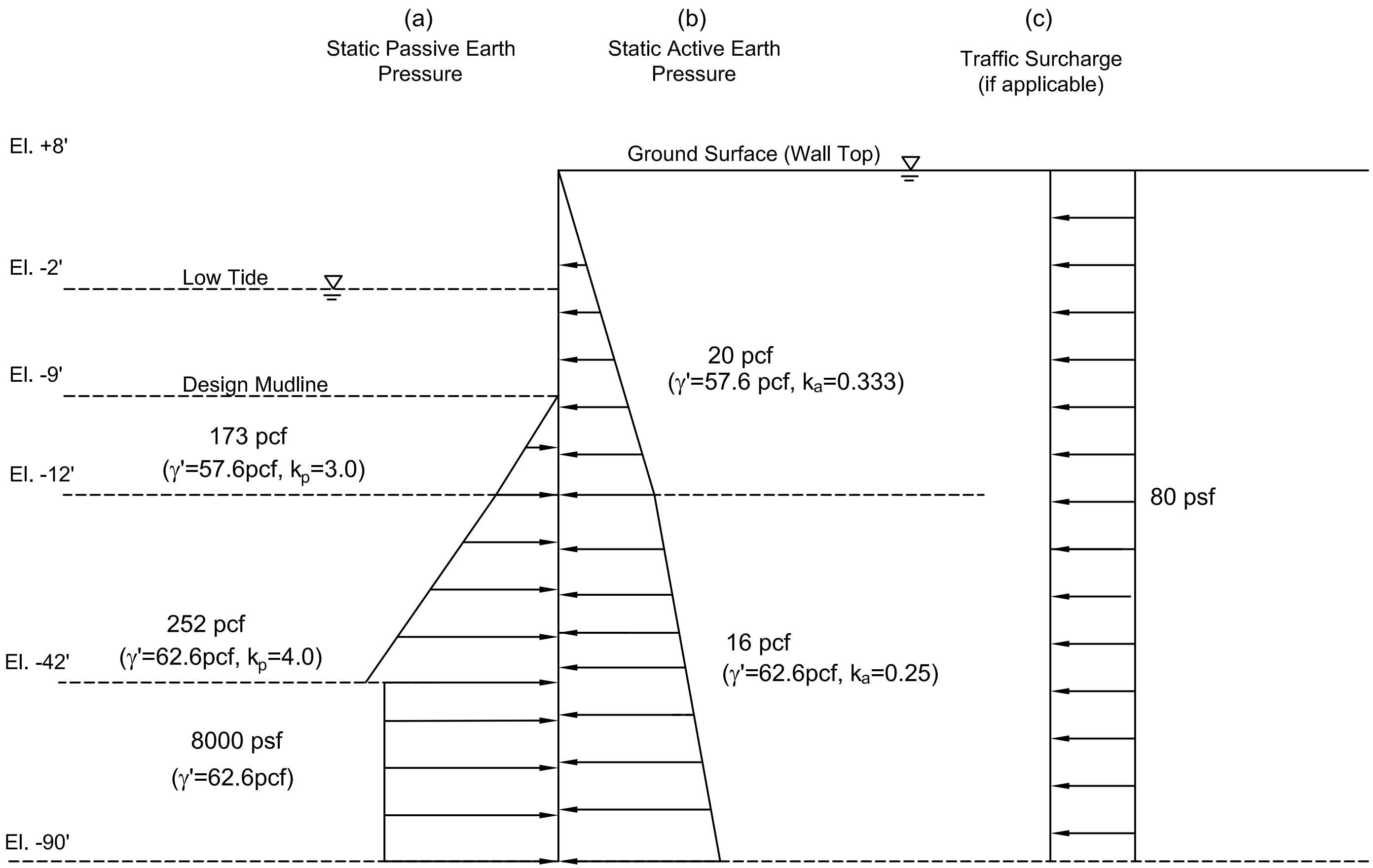
11.2.1 Lateral Earth Pressures

The lateral pressure diagrams for the seawalls are shown in Figure 7 and Figure 8 for the static and seismic conditions, respectively. The walls are assumed to be free to move laterally at the top and under undrained condition. In addition to the pressures shown in Figure 7 and Figure 8, the hydraulic static pressure of 62.4 psf should be added.

The seismic earth pressures were estimated following Caltrans design criteria using one third of Caltrans PGA of 0.49g. The seismic incremental lateral earth pressure of 35 psf per foot can be used as shown in Figure 8, which is a larger of the seismic earth pressures due to 30 degree of sand materials and due to 200 psf of undrained shear strength of liquefiable soil.

11.2.2 Passive Resistance

The lateral passive diagrams along the seawall are shown in Figure 7 and Figure 8. The undrained shear strength is used for the liquefiable soils in front of the seawalls. The full passive resistance is expected to be mobilized at a horizontal movement of 5 percent of the embedment depth, measured from the lowest adjacent grade to the bottom of the pile/wall.



Notes:

(1) The above earth pressure can be directly used for sheet pile walls.

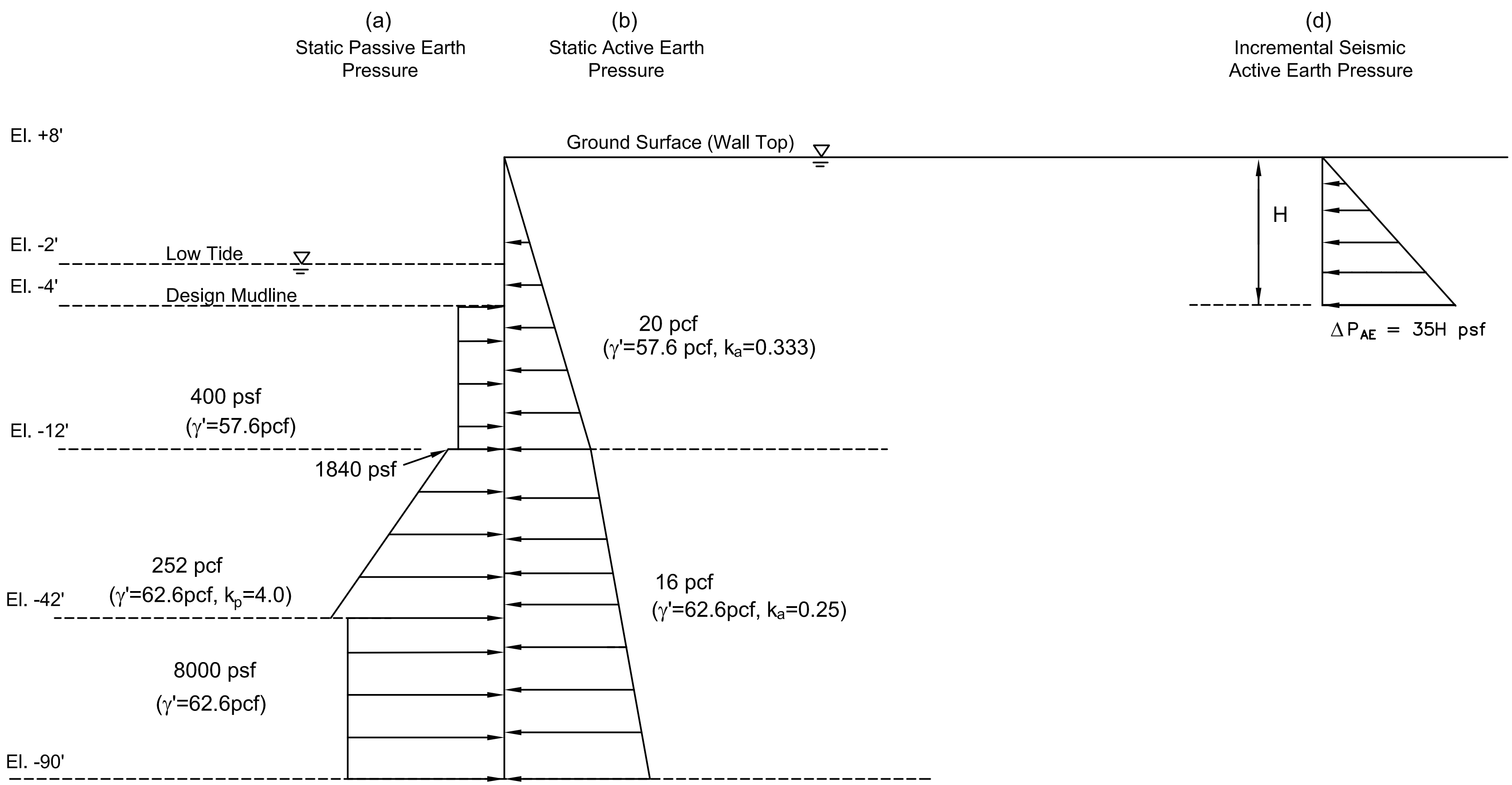
(2) For the king piles with sheet piles,

(2a) Above design grade, multiply active earth pressure and traffic surcharge (if applicable) by center-to-center king pile spacing.

(2b) Below design grade, multiply active earth pressure and traffic surcharge (if applicable) by king pile size.

(2c) Below design grade, multiply passive earth pressure by king pile size with a arching factor of pile spacing to pile size (up to 2.5).

Earth Mechanics, Inc. Geotechnical and Earthquake Engineering	Collins Island Bridge and Seawalls	Lateral Earth Pressure Diagram for Seawalls (Static Condition)
	Project No. 23-115 / Date: October 2023	FIGURE 7



Notes:

(1) The above earth pressure can be directly used for sheet pile walls.

(2) For the king piles with sheet piles,

(2a) Above design grade, multiply active earth pressure and traffic surcharge (if applicable) by center-to-center king pile spacing.

(2b) Below design grade, multiply active earth pressure and traffic surcharge (if applicable) by king pile size.

(2c) Below design grade, multiply passive earth pressure by king pile size with a arching factor of pile spacing to pile size (up to 2.5).

Earth Mechanics, Inc. Geotechnical and Earthquake Engineering	Collins Island Bridge and Seawalls	Lateral Earth Pressure Diagram for Seawalls (Seismic Condition)
	Project No. 23-115 / Date: October 2023	FIGURE 8

12.0 NOTES FOR SPECIFICATIONS AND CONSTRUCTION

12.1 Earthwork

Earthwork should be performed in accordance with Section 19 of the Caltrans Standard Specifications (2023b). Appropriate measures should be taken to prevent damage to adjacent existing structures and utilities. Any design and construction of temporary sloping, sheeting, or shoring should be made the contractor's responsibility. It should be noted that it is the responsibility of the contractor to oversee the safety of the workers in the field during construction. The contractor shall conform to all applicable occupational and health standards, rules, regulations, and orders established by the State of California. In addition, other State, County, or Municipal regulations may supersede the recommendations presented in this section. If a trench shoring design and safety plan is required, the geotechnical consultant should review the plan to confirm that recommendations presented in this report have been applied to the design.

12.2 Groundwater Control

The high tide water elevation is located at El. +7.7 feet as shown in the general plan. Groundwater will be encountered during construction of the CIDH piles. Contractor should be fully prepared for a wet construction when bidding and selecting construction equipment and methods.

12.3 CIDH Pile Construction

Construction of CIDH piles should follow Section 49-3.02 of the Caltrans Standard Specifications (2023b). Per Caltrans Memo To Designers 3-1 (2014b), a minimum of 3-inch of concrete cover over reinforcement should be provided to improve the construction of the 24-inch diameter CIDH piles. Very challenging CIDH pile construction is anticipated due to a wet construction and high groundwater. The project site is also located in a tidal zone and marine environment. Difficult drilling conditions are anticipated because the project site is underlain by saturated, caving soils with localized dense and hard soil layers. The bedrock has high SPT blowcounts; drilling and excavating are anticipated to be slow and difficult.

For a wet pile construction, the contractor is required to maintain a minimum 10-foot head of slurry over the piezometric surface at all times during CIDH pile construction. This minimum head of slurry is required to prevent a "quick" condition during the CIDH pile excavation. Water is not allowed as slurry, even if full length casing is used during pile excavation. As a standard Caltrans practice for "wet" construction, PVC tubings must be installed within the reinforcement cage of the CIDH pile for gamma-gamma testing per Caltrans Memo-To-Designers 3-1 (2014b).

Soil caving can be controlled using a temporary casing or slurry. The use of temporary casing is left to the contractor's discretion. Temporary casings should have an outer diameter equal to or exceeding the pile diameter, and should be placed tight in hole. Temporary casing installation may be difficult due to the presence of dense and hard soil layers. The temporary casing should be pulled as the concrete is being poured while always maintaining at least a 5-foot head of concrete inside the temporary casing.

The Contractor should be required to drill the bottom of the shaft boring with a clean-out bucket to ensure adequate removal of loose soils. The shaft borings should be inspected and approved by the Resident Engineer prior to installation of reinforcement. Extreme care in drilling, placement of steel, and the pouring of concrete is essential to avoid excessive disturbance of pile boring walls. Concrete placement by pumping or tremie tube to the bottom of the pile borings will be required. Sufficient space should be provided in the pile reinforcing cage during fabrication to allow the insertion of a tremie tube for concrete placement.

The pile reinforcing cage should be installed and the concrete pumped, immediately after drilling is completed. No borings should be drilled immediately adjacent to another pile until the concrete in the other pile has attained its initial set.

In the event that any boring becomes bell-shaped and cannot be advanced due to severe caving, all loose material should be removed from the bottom of the boring and the caved region filled with a low strength sand-cement slurry. Drilling may continue when the slurry has reached its initial set.

The above information is not meant to direct the pile contractor to excavate and build the CIDH piles; any construction means and methods remain the responsibility of the pile contractor.

12.4 Sheet Piling

Piles should be installed in accordance with Section 49-2 of the Caltrans Standard Specifications (2023b). Piles should be driven at least to the specified tip elevation shown. Piles that are materially out of line should be removed and re-driven or replaced.

Contractors should review the LOTB sheet (Appendix A) and follow the requirements in the Caltrans Standard Specification (2023b) in selecting the pile driving equipment.

12.5 Review of Construction Plans

Recommendations contained herein are based on current design information. The geotechnical consultant should review the final construction plans and specifications in order to confirm that the general intent of the recommendation contained in this report have been incorporated into the final construction documents. Recommendations presented in this report may require modification or additional recommendations may be necessary based on the final design.

12.6 Geotechnical Observation and Testing

Qualified geotechnical personnel should perform inspections and testing during the following stages of construction:

- Shoring installation, if necessary.
- Construction of CIDH piles.
- Construction of sheet piles / king piles with sheet piles.
- When any unusual subsurface conditions are encountered.

13.0 REFERENCES

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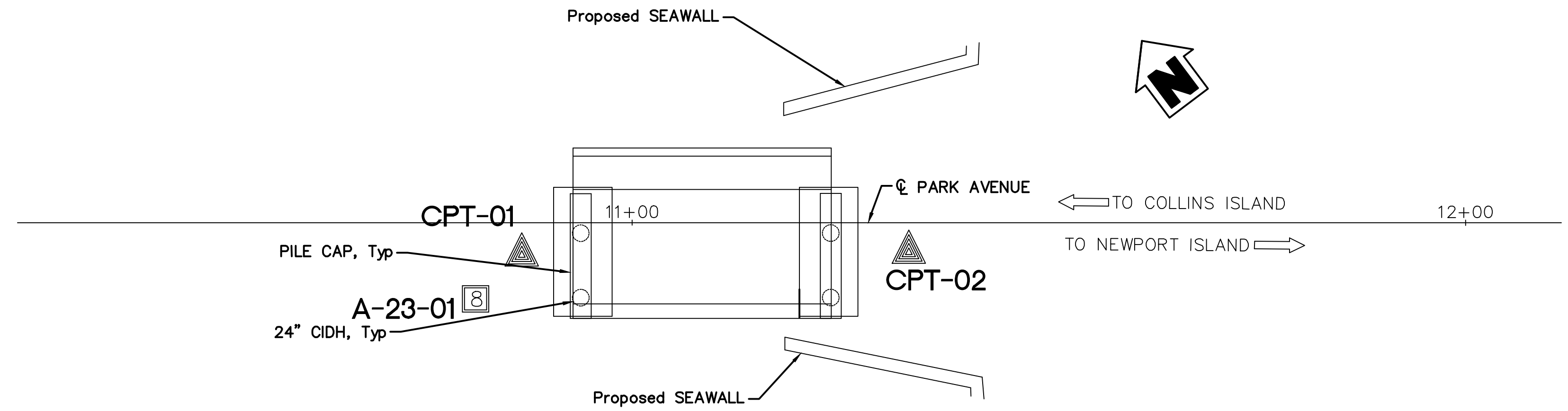
APPENDIX A

LOG-OF-TEST-BORING SHEET

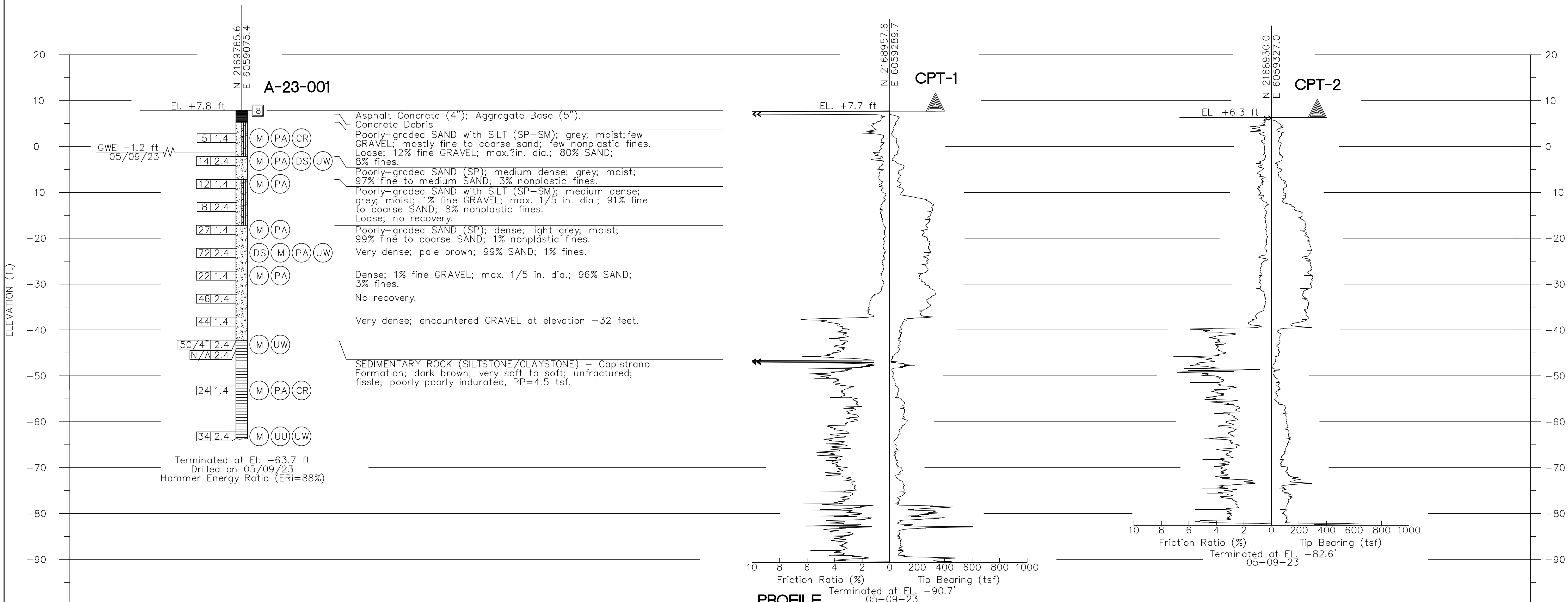
NOTES:

(1) This LOTB sheet (Boring Record) was prepared in accordance with Caltrans Soil and Rock Logging, Classification and Presentation Manual (2010).
(2) 1.4" samples were taken using a Standard Penetration Sampler and 2.4" samples were taken using a California Modified Sampler.
(3) An automatic trip hammer system consisting of a hammer weight of 140 lbs falling a distance of 30" was used to advance samplers.
(4) For Soil Legend, see Caltrans Standard Plans A10F and A10G .
(5) Strong odor of petroleum/hydrocarbon was encountered in the soil samples collected through out the project site.



PLAN

1" = 10'



PROFILE

VERTICAL 1" = 10'

Earth Mechanics, Inc. Geotechnical & Earthquake Engineering 17800 NEWHOPE STREET, SUITE B, FOUNTAIN VALLEY, CA 92708. P: 714-751-3826	REGISTERED PROFESSIONAL ENGINEER ALAHESWARAN THURAIRAJAH NO. GE 3123 EXP. 9-30-2023 GEOTECHNICAL STATE OF CALIFORNIA	REVIEWED: (ALAHESH) A. THURAIRAJAH GE 3123 DESIGN ENGINEER'S NAME DESIGNED: MH DRAWN: MH CHECKED: AT DATE: 10/18/23

NO.	DATE	DESCRIPTION OF REVISIONS	APPROVED

REVIEWED:

[NAME] RCE XXXXX
CITY PROJECT MANAGER

DATE ______

LOG OF TEST BORING 1 OF 1
COLLINS ISLAND BRIDGE

CITY OF NEWPORT BEACH
PUBLIC WORKS DEPARTMENT

X-XXXX-X
SHEET XX OF XX

X:\Projects\2023\23-115 - MBI, Collins Island Bridge\Field Investigation\LOTB\CollinsIsland_LOTBs_rev.dwg 10/23/23 - 9:59am M.Hoshiyama

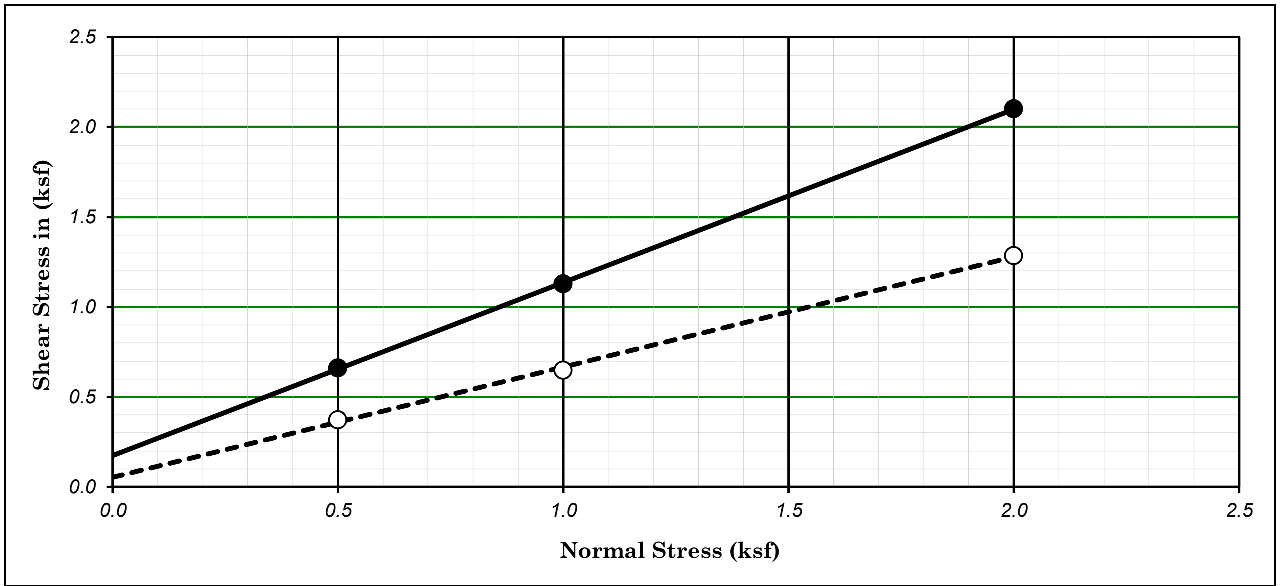
APPENDIX B

LABORATORY TEST RESULTS

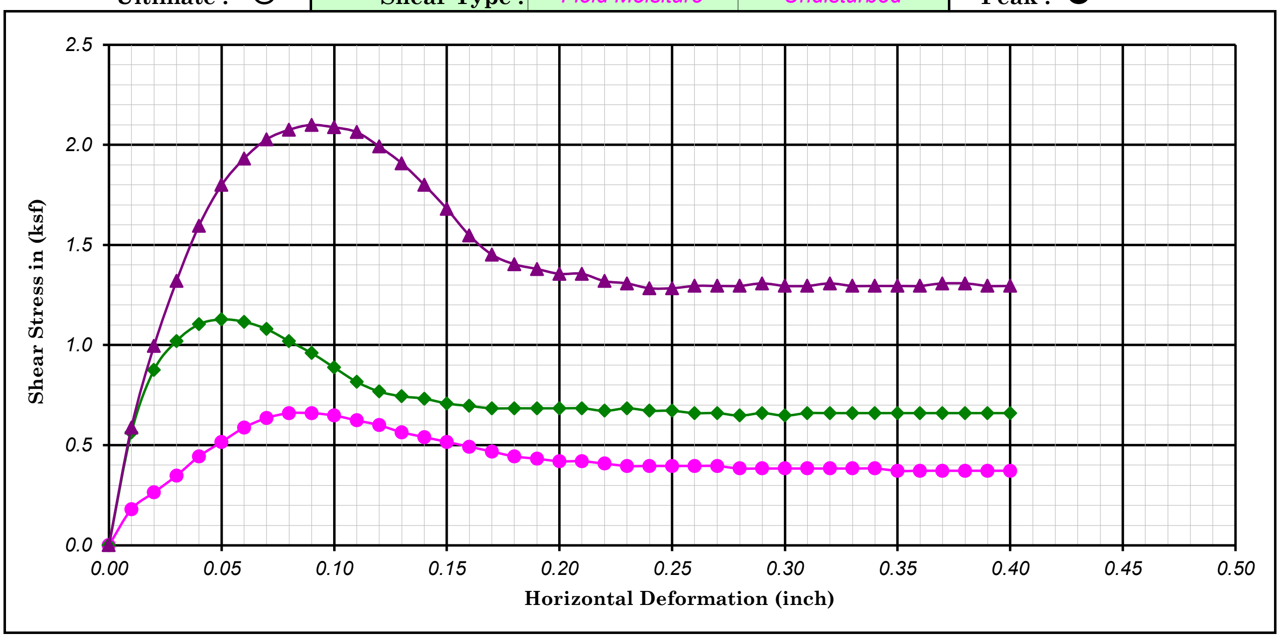
TABLE B-1 SUMMARY OF LABORATORY TEST RESULTS

Project No. : *23-115* **Project Name :** MBI, Collins Island Bridge

Boring No .	Sample No.	Sample Depth (ft)	Soil Identification (group symbol) ASTM D2488/D2487	Moisture Content ASTM D2216 (%)	Total Unit Weight ASTM D2937 (pcf)	Pocket Penetrometer (tsf)	Torvane Shear (tsf)	Grain Size Distribution GR:SA:FI (%)	Sand Equivalent (CT-217)	Atterberg Limits ASTM D4318 (LL/PL/PI)	Soil-Minimum Resistivity CT-643 (ohm-cm)	Soil-pH CT-643	Soil-Soluble Sulfate Content CT-417 (ppm)	Soil-Moisture Free Chloride Content CT-422 (ppm)
A-23-001	*S-1*	5	SP-SM	30.6				12:80:8			140	8.3	1680	1030
A-23-001	*D-2*	10	SP	20.7	125.8			0:97:3						
A-23-001	*S-3*	15	SP-SM	25.5				1:91:8						
A-23-001	*S-5*	25	SP	18.9				0:99:1						
A-23-001	*D-6*	30	SP	17.3	125.1			0:99:1						
A-23-001	*S-7*	35	SP	17.4				1:96:3						
A-23-001	*D-10*	50	SP-SM to CL	20.7	116.6	>4.5								
A-23-001	*S-11*	60	OH	52.2						89/52/37	91	7.3	4068	565
A-23-001	*D-12*	70	OH	71.0	90.5	>4.5								



Ultimate : ○	Shear Type :	Field Moisiture	Undisturbed	Peak : ●



Boring No.	: A-23-001		**Strength Intercept (C) :**	0.17	(ksf)	**Peak**	0.05	(ksf)
Sample No.	: D-2			8.33	(kPa)		2.59	(kPa)
Depth (ft/m)	: 10.0	0.00	**Friction Angle (ϕ) :**	43.88	Degree		31.47	Degree

Ultimate

Description : Dark gray, Poorly graded SAND (SP) — **Shear Rate** (inch/minute) : 0.02

SYMBOL	MOISTURE CONTENT (%)	DRY DENSITY		VOID RATIO	NORMAL STRESS		PEAK STRESS		ULTIMATE STRESS	
		(pcf)	(kN/m^3)		(ksf)	(kPa)	(ksf)	(kPa)	(ksf)	(kPa)
●	20.67	105.25	16.57	0.60	0.50	23.94	0.66	31.60	0.37	17.81
◆	20.95	105.15	16.55	0.60	1.00	47.88	1.13	54.01	0.65	31.03
▲	21.00	107.10	16.86	0.57	2.00	95.76	2.10	100.55	1.28	61.48

Earth Mechanics, Inc.

Geotechnical and Earthquake Engineering

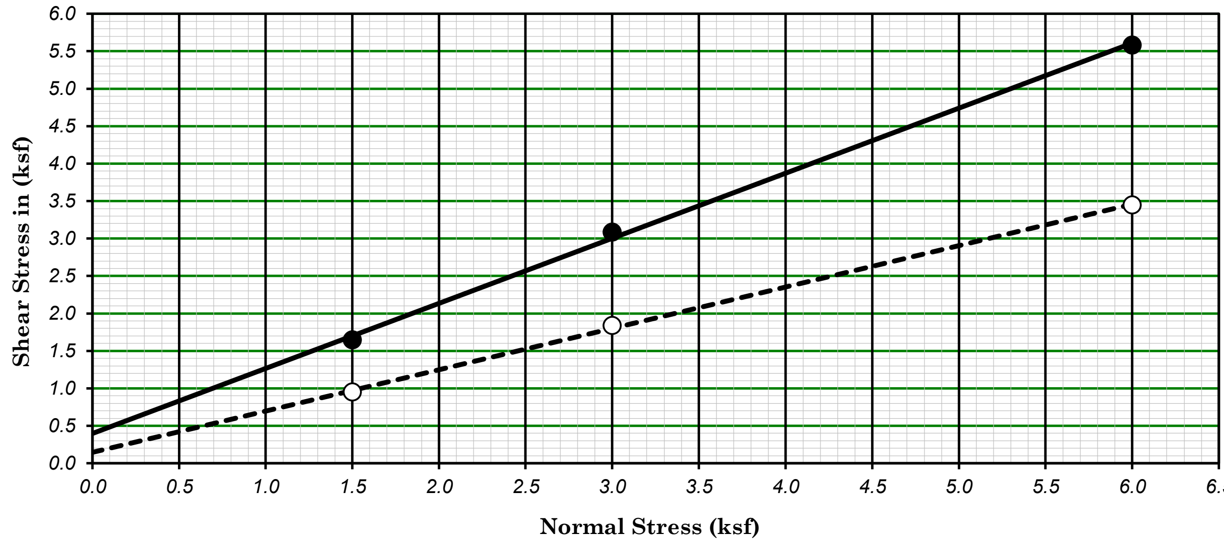
MBI, Collins Island Bridge

Project No. : 23-115 **Date :** 05/22/23

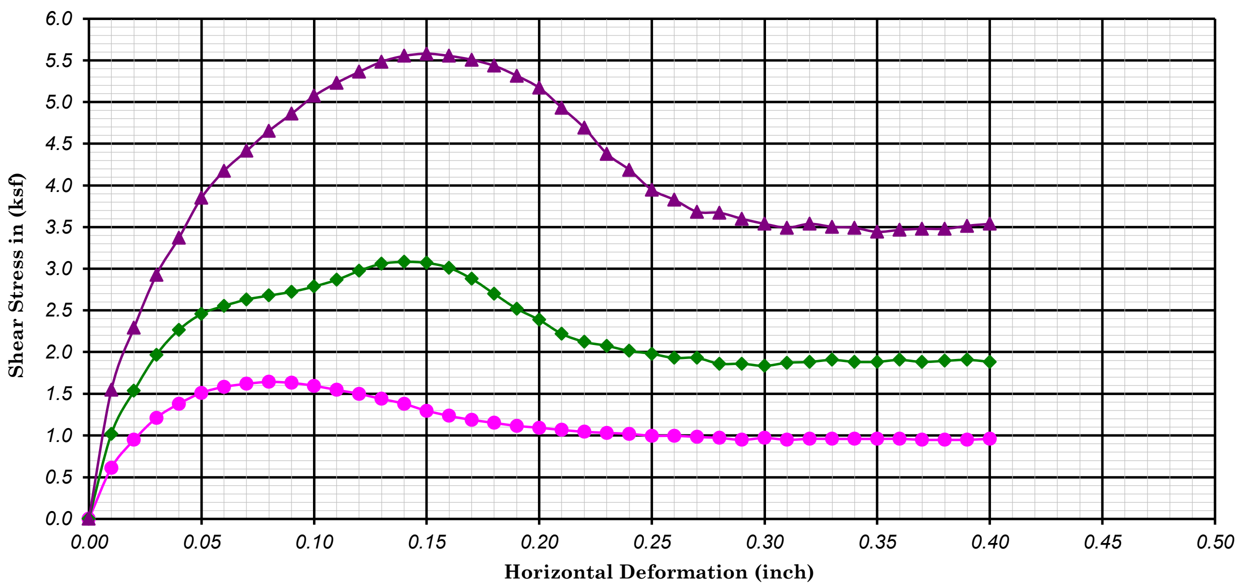
DIRECT SHEAR TEST

(ASTM D-3080)

Figure No.



Ultimate : ○ | Shear Type : Field Moisture | Undisturbed | Peak : ●



Boring No.	: A-23-001	Strength Intercept (C) :	0.40	(ksf)	Peak	0.14	(ksf)	Ultimate
Sample No.	: D-6		18.96	(kPa)		6.89	(kPa)	
Depth (ft/m)	: 30.0 / 0.00	Friction Angle (ϕ) :	40.98	Degree		28.90	Degree	
Description	: Brown, Poorly graded SAND (SP)					Shear Rate (inch/minute) :	0.02	

SYMBOL	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	DRY DENSITY (kN/m^3)	VOID RATIO	NORMAL STRESS (ksf)	NORMAL STRESS (kPa)	PEAK STRESS (ksf)	PEAK STRESS (kPa)	ULTIMATE STRESS (ksf)	ULTIMATE STRESS (kPa)
●	17.26	104.28	16.41	0.62	1.50	71.82	1.64	78.71	0.95	45.39
◆	17.14	104.92	16.51	0.61	3.00	143.64	3.08	147.66	1.84	87.91
▲	17.38	107.54	16.93	0.57	6.00	287.28	5.58	267.17	3.44	164.90

Earth Mechanics, Inc.

Geotechnical and Earthquake Engineering

MBI, Collins Island Bridge

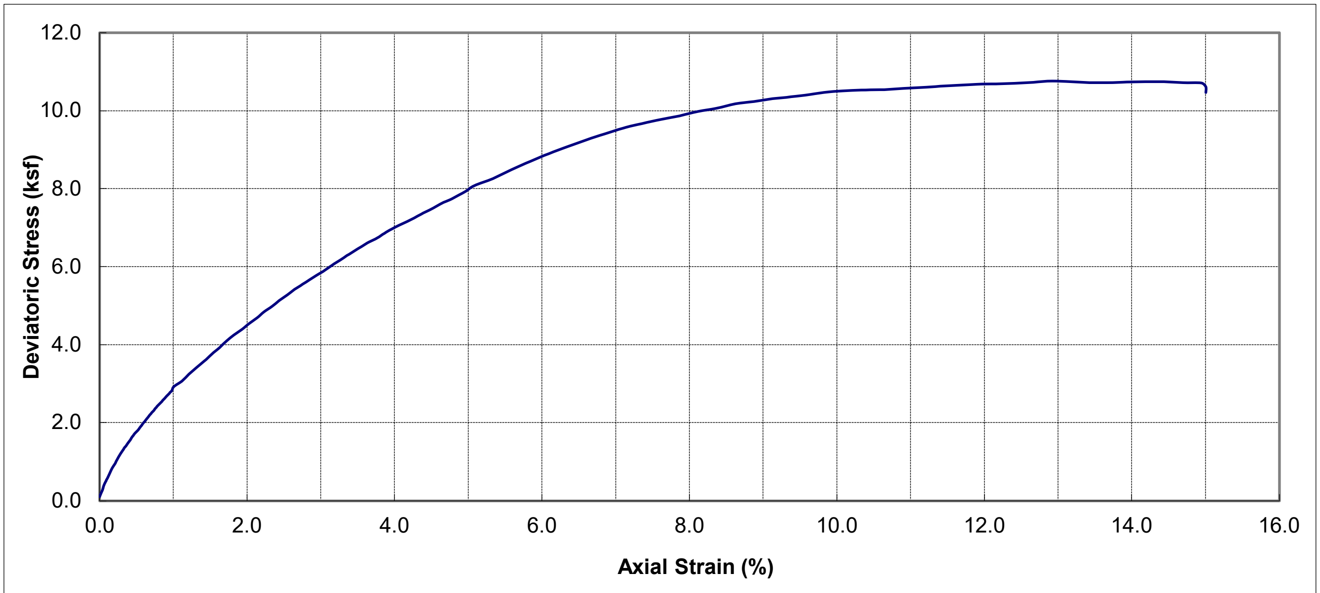
DIRECT SHEAR TEST

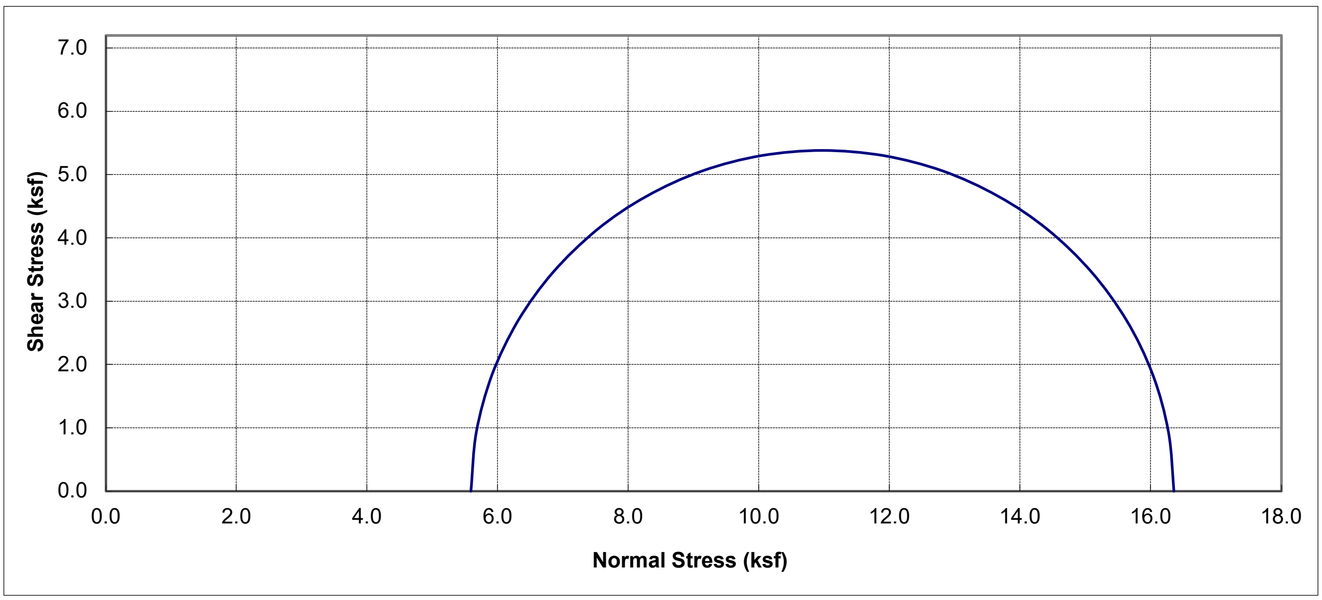
(ASTM D-3080)

Project No. : 23-115

Date : 05/22/23

Figure No.





Boring No.	Sample No.	Depth (ft)	Soil Type	Dry Density (pcf)	Moisture Content (%)	Conf. Stress (ksf)	Max Dev. Stress (ksf)	Initial Saturation (%)
A-23-001	D-12	70	Dark olive, Organic SILT (MH)	54.5	70.96	5.59	10.76	91.7

Earth Mechanics, Inc. Geotechnical and Earthquake Engineering	**MBI, Collins Island Bridge**
Project No. : 23-115 **Date :** 05/23/23	**UNCONSOLIDATED UNDRAINED TEST** **(ASTM D2850)** Figure No. :

Michael Baker
INTERNATIONAL

October 11, 2023

Robert Stein, PhD, PE
Assistant City Engineer
City of Newport Beach
100 Civic Center Drive
Newport Beach, CA 92660

RE: PALEONTOLOGICAL RESOURCES ASSESSMENT FOR THE COLLINS ISLAND BRIDGE REPLACEMENT PROJECT, NEWPORT BEACH, ORANGE COUNTY, CALIFORNIA

Dear Dr. Stein:

In support of the proposed Collins Island Bridge Replacement Project (project), Michael Baker International staff conducted a fossil locality search at the Natural History Museum of Los Angeles County (NHMLAC), literature and geologic map review, and a paleontological resources sensitivity analysis. These efforts identified the paleontological sensitivity of the project area and determined whether the project could result in significant impacts to paleontological resources in accordance with the California Environmental Quality Act (CEQA). Methods, results, and recommendations are summarized below; figures are provided in **Attachment 1**.

PROJECT SITE

The project site is located in the City of Newport Beach in Orange County, California. The project site is the Waters Way Bridge (No. 55C-0265), colloquially known as the Collins Island Bridge, and its immediate vicinity on Balboa Island in Newport Bay. Collins Island is located on the western tip of Balboa Island and is connected to the greater Balboa Island via the Collins Island Bridge. Regional access to the project site is provided via State Route 1 (SR-1; Pacific Coast Highway) and local access to the site is provided via Marine Avenue (across the Balboa Island North Channel), and North Bay Front and Park Avenue on Balboa Island (**Figure 1**). The project site is within Section 35 of Township 6 South and Range 10 West, San Bernardino Baseline and Meridian of the Newport Beach OE S, California 7.5-minute US Geological Survey (USGS) topographic quadrangle (**Figure 2**).

PROJECT DESCRIPTION

The project includes three major components: 1) bridge replacement, 2) seawall improvements, and 3) future pump station accommodations.

Bridge Replacement: The proposed new bridge would be designed to be a total of 20 feet and 6 inches in width to accommodate one vehicle travel lane that is 13 feet and 9 inches wide, one 4-foot-wide sidewalk, and concrete barriers on each side to provide protection from projected sea level rise. The bridge would be 31 feet in length spanning over existing concrete sheet pile bulkheads. The existing bridge slope along the roadway and sidewalk bridge approaches on both sides of the bridge exceed 5 percent. Therefore, the project includes adjusting the profiles to comply with Americans with Disabilities Act (ADA) standards. Landscaped areas and the bridge monument would be improved to increase sight distance along the adjacent walkways to increase pedestrian safety. A new stop sign and limit line would be added at the intersection on both ends of the bridge.

Additionally, street, sidewalk, and landscaping improvements are proposed on the Balboa Island side along the Bay Front sidewalk and Park Avenue eastward until the alley. Anticipated improvements include monument sign construction, irrigation, paving, and landscaping.

Seawall Improvements: The project includes increasing the height of existing seawalls adjacent to the bridge to protect properties from water levels associated with high tides and storm surges and anticipated future water surface elevation increases due to climate change. Currently, most seawalls along Collins Island Bridge and the Bay Front sidewalk consist of concrete sheet pile bulkheads with a concrete cap (coping) elevation of approximately 9 feet North American Vertical Datum of 1988 (NAVD 88). The proposed seawall improvements would be designed to have a top of wall coping elevation of 11 feet NAVD 88 with a future cap extension elevation up to 14 feet NAVD 88.

To maintain consistency between Collins Island and Balboa Island, existing seawalls along the Bay Front sidewalk would also be improved to meet ADA requirements and to accommodate future sea level rise. The Bay Front sidewalks adjacent to the new proposed seawalls would be raised to provide a minimum of 42 inches from sidewalk to top of coping.

The new seawalls would be designed to allow access to existing boat ramps and docks. However, certain docks would be temporarily relocated during construction activities. Where possible, the existing concrete sheet pile bulkhead system would remain in place to reduce disturbance and associated environmental impacts. In the case of Bay Front sidewalk seawall improvements, new steel sheet piles would be placed seaward from the existing concrete sheet piles. A new sidewalk and parapet cap would provide seawall protection.

Future Pump Station Accommodations: The City is currently in the process of designing a new stormwater pump station on Park Avenue near the Collins Island Bridge as part of a separate project. The pump station is designed to have discharge outlets located near the east abutment of the Collins Island Bridge (Waters Way Bridge [No. 55C-0265]). As such, given that the project and pump station project are being designed concurrently, the project includes pump station accommodations to convey anticipated stormwater outflow into the bay adjacent to the new bridge. Specifically, weir structures would be constructed adjacent to the proposed seawalls along the east abutment of the bridge to control the rate of stormwater outflow. In addition, portions of the future pump station outlet pipes that connect to the weir structure are proposed within this project. Two outlet pipes are proposed on the northern side of the bridge and two outlet pipes are proposed on the southern side of the bridge. It should be noted that while the pump station project is being designed by the City concurrently with the project, the pump station project is not a part of the project and would be approved separately.

GEOLOGIC SETTING

California is divided into 11 geomorphic provinces, each defined by unique geologic and geomorphic characteristics. The project site is located along the central coastal portion of the Peninsular Ranges geomorphic province, distinguished by northwest-trending mountain ranges and valleys following the branching San Andreas fault. This geomorphic province also includes physiogeographic features such as the Los Angeles Basin, the southern members of the Channel Islands, and the continental shelf (CGS 2002). The Peninsular Ranges province crosses several counties, as well as Baja California. The Pacific Ocean borders it to the west, the Transverse Ranges geomorphic province to the north, and the Colorado Desert geomorphic province to the east. The Peninsular Ranges batholith dominates the Peninsular Ranges.

The geology of Newport Beach has been mapped by Morton and Miller (2006) at a scale of 1:100,000. Geologic units underlying the project area have been mapped as late Holocene-aged very young estuarine deposits (Qes of Morton and Miller 2006). Deposits from the Holocene epoch (less than 11,700 years ago) can contain remains of animals and plants; however, only those from the middle to early Holocene (older than about 5,000 radiocarbon years) are considered scientifically important or significant (SVP 2010). Less than 3 miles from the project site, Pliocene- to Pleistocene-age localities have also been mapped (Palos Verdes Sand and Fernando Formation) (**Tables 1** and **2**). Soils of the project site are mapped as Beach sand (hclq) (NRCS 2023).

PALEONTOLOGICAL RESOURCES IDENTIFICATION METHODS

The records search results, literature review, and paleontological sensitivity analysis are presented below.

RECORDS SEARCHES AND LITERATURE REVIEW

The NHMLAC completed a paleontology collection records search for locality and specimen data on August 20, 2023. The results of that search are included in **Attachment 2**. The records search identified 12 known fossil localities in the NHMLAC's collection in the vicinity of the project site (**Table 1**). Pliocene- and Pleistocene-age marine deposits have yielded scientifically important fossils, including sharks, ducks, horses, mammoths, and invertebrates, within 3 miles of the project site.

Table 1: NHMLAC Paleontological Records Search Results

Locality Number	Distance to Project Site	Formation	Taxa	Depth
LACM VP 5466	~ 0.5 miles NE	Palos Verdes Sand (Pleistocene)	Horse (*Equus*)	Unknown
LACM IP 25839	~ 0.5 miles NE	Palos Verdes Sand (Pleistocene)	Invertebrates including: *Eulithidium, Flossaulax, Ostrea, Donax, Callianax, Mitrella, Chama, Balcis, Strioterebrum, Melampus, Amphissa, Aesopus, Laevicardium, Bulla, Crepidula, Dentalium, Leptopecten, Tellina*	Unknown
LACM IP 31435, 4760	~ 0.75 miles SE	Palos Verdes Sand (Pleistocene)	Invertebrates including: bivalves (*Leukoma, Ostrea, Crepidula, Saxidomus, Tresus, Leptopecten, Macoma, Lucinisca, Brachidontes, Tellina*); gastropods (*Caesia, Pseudomelatoma, Callianax*); barnacle (Balanidae); scaphopod (*Dentalium*)	Unknown
LACM VP 4254; LACM IP 17103, 17104	~ 1.5 miles E	Palos Verdes Sand (Pleistocene)	Duck (*Chendytes*); saltwater clam (*Chama, Septifer, Epilucina*); mussel (*Mytilus*); sponge trace (*Entobia*); feeding trace (*Oichnus*); oyster (*Ostrea*); turban snail (*Megastraea*); limpet (*Lottia*)	Unknown
LACM VP 3408	~ 2 miles NE	Fernando Formation (Pliocene to Pleistocene)	Shark (*Charcharodon, Carcharocles, Alopias*); fragmentary marine mammal bones	Unknown
LACM VP 3407, 4426; LACM IP 71, 5627	~ 3.3 miles NE	Palos Verdes Sand (Pleistocene)	Mammoth (*Mammuthus*); uncatalogued birds, fish, mammals, and invertebrates	Surface

Formation ages from National Geologic Map Database (2023)

Additionally, Michael Baker International conducted a supplemental investigation within 3 miles of the project site using the following online sources:

- University of California Museum of Paleontology Locality Search (UCMP 2023)

- San Diego Natural History Museum Collection Database (SDNHM 2023)
- The Paleobiology Database (PBDB 2023)

The supplemental investigation identified no additional fossil-bearing localities within the project site. Fourteen localities from Pleistocene-aged geologic formations (Palos Verdes Sand or unknown sediments) were identified within 3 miles of the project site. Nine additional localities from the Palos Verdes Sand have been recorded in the SDNHM database, though their exact distance to the project site is unknown. The UCMP database records numerous localities from Holocene and Recent Quaternary sediments using search terms such as "Balboa Bay" and "Newport Beach," though their exact distance to the project site is unknown. The records searches were limited to data available online (**Table 2**).

Table 2: Supplemental Paleontological Records Search

Locality Number	Distance to Project Site	Formation	Taxa	Depth
81919 (PBDB)	~0.5 miles W	Unknown (late Pleistocene)	*Eumetopias jubatus* (eared seal)	Unknown
160361, 219032 (PBDB)	~1 mile SE	Palos Verdes Sand (late Pleistocene)	Fish (sharks, wrasses, sculpins, cusk-eels, perches), Invertebrates (barnacles, crabs, polychaete worms brachiopods, bryozoans, sea urchins, sea stars, bivalves, gastropods, scaphopods, chitons)	Unknown
219029, 219030 (PBDB)	~1.5 miles N	Palos Verdes Sand (late Pleistocene)	Fish (sharks), Invertebrates (bryozoans, crabs, bivalves, gastropods, scaphopods, chitons)	Unknown
226458 (PBDB)	~2 miles W	Palos Verdes Sand (late Pleistocene)	Crabs	Unknown
96930, 96935–96939 (PBDB)	~2 miles NE	Unknown (late Pleistocene)	Invertebrates (bivalves, gastropods, barnacles, sea stars, scaphopods)	Unknown
219855, 226567 (PBDB)	~3 miles NE	Unknown (Pleistocene)	Cancridae (crabs), Myctophidae (lLanternfish)	Unknown
P1268 (UCMP)	Unknown (Balboa Bay)	Unknown Holocene sediments	Pollen samples	Unknown

Locality Number	Distance to Project Site	Formation	Taxa	Depth
E6100, 6127, 6210, 6211, 6215, 6216, 6458, 6512, 8281 (UCMP)	Unknown (Balboa Bay or Balboa)	Recent Quaternary sediments	None currently identified	Unknown
RS6706 (UCMP)	Unknown (Balboa)	Recent Quaternary sediments	*Bittium quadrifilatum* (sea snail)	Unknown
RS9328 (UCMP)	Unknown (Balboa)	Recent Quaternary sediments	*Barnea subtruncata* (bivavle)	Unknown
A3991, B6865, E1662, E477, E6161, E6213, E6350, E6436, E8328, E8853, E9761, IP10203, IP14860, RS11091, RS2480, RS2534, RS5197, RS5906, RS6010, RS719, RS7747, RS779, RS789, RS9242 (UCMP)	Unknown (Newport Beach, Newport Bay, or Newport Harbor)	Recent Quaternary sediments	Gastropods, corals, bivalves	Unknown
2619, 2726, 2992, 5276–5281 (SDNHM)	Unknown (Newport Beach)	Palos Verdes Sand (Pleistocene)	Numerous invertebrates (bivalves, gastropods, echinoderms, barnacles, scaphopods, bryozoans, forams)	Unknown

Paleontological Resources Sensitivity Analysis

The NHMLAC paleontological records search and fossil locality searches in online databases (PBDB, SDNHM, and UCMP) did not identify any paleontological resources within the project site. Several localities have been found within 3 miles of the project site; however, these localities are from rock formations (Pleistocene Palos Verdes Sand and Fernando Formation deposits) older than those mapped as underlying the project site. One locality of Holocene age, equivalent to sediments underlying the project site, was found within 3 miles of the project site (UCMP 2023). Per mitigation impact guidelines set forth by the Society of Vertebrate Paleontology (SVP 2010), due to the fossil sensitivity of the rock formations present within the project site, the project has a low potential to disturb paleontological resources within undisturbed sedimentary deposits and bedrock.

RECOMMENDATIONS

The following mitigation measure (MM) is recommended to be implemented in the event of any discovery of unknown paleontological resources during earthwork.

MM PALEO-1: Paleontological Resources Inadvertent Discovery. In the event that paleontological resources are encountered during earth-disturbing activities, all construction activities within 100 feet of the discovery shall be temporarily halted until a qualified paleontologist shall evaluate the findings and make a recommendation. The assessment will follow Society of Vertebrate Paleontology (SVP) standards as delineated in the *Standard Procedures for the Assessment and Mitigation of Adverse Impacts to Paleontological Resources* (2010). If the qualified paleontologist finds that the resource is not a significant fossil, then work may resume immediately. If the qualified paleontologist finds the resource is potentially significant, then the qualified paleontologist shall make recommendations for appropriate treatment in accordance with SVP guidelines for identification, evaluation, disclosure, avoidance, recovery, and/or curation, as appropriate. The City shall determine the appropriate treatment of the find. Work cannot resume within the no-work radius until the City, through consultation as appropriate, determines that appropriate treatment measures have been completed to the satisfaction of the City. Any fossils recovered during mitigation shall be cleaned, identified, catalogued, and permanently curated with an accredited and permanent scientific institution with a research interest in the materials, such as the Cooper Laboratory in Santa Ana.

A qualified professional paleontologist is a professional with a graduate degree in paleontology, geology, or related field, with demonstrated experience in the vertebrate, invertebrate, or botanical paleontology of California, as well as at least one year of full-time professional experience or equivalent specialized training in paleontological research (i.e., the identification of fossil deposits, application of paleontological field and laboratory procedures and techniques, and curation of fossil specimens), and at least four months of supervised field and analytic experience in general North American paleontology as defined by the SVP.

PREPARER QUALIFICATIONS

This memorandum was prepared by Michael Baker International Senior Paleontologist Peter Kloess, PhD. Senior Cultural Resources Manager Margo Nayyar reviewed the memo for quality control.

Peter A. Kloess, PhD, Principal Investigator—Paleontology is a principal investigator and paleontologist with over 20 years of experience in paleontology, with 8 years in paleontology mitigation. His experience includes private and public consultation, field monitoring, excavation, and laboratory research on projects across the western United States, predominantly in California. He has consulting experience with a range of projects, including construction, transportation, utility, transmission, monitoring, and surveys, as well as expertise recovering a diversity of fossils from project sites, such as marine invertebrates, microfossils, plants, small mammals, and birds, large marine and terrestrial mammals, and dinosaurs. He also has extensive experience in paleontological museum collections and lab settings. He has worked on and co-led scientific excavations of large mammals and dinosaurs in California, Utah, New Mexico, and Montana. Mr. Kloess has served as a lab preparator and assistant curator for paleontology museums in California and Montana, where his duties included manual preparation of specimens, casting, jacketing, public outreach, cataloging, and curation. He meets the Society of Vertebrate Paleontology's standards for paleontological Principal Investigator.

Margo Nayyar, MA, is a senior architectural historian with 13 years of cultural management experience in California, Nevada, Arizona, Texas, Idaho, and Mississippi. Her experience includes built environment surveys, evaluation of historic-era resources using guidelines outlined in the California and National Registers, and preparation of cultural resources technical studies pursuant to CEQA and NHPA Section 106, including identification studies, finding of effect documents, memorandum of agreements, programmatic agreements, and Historic American Buildings Survey/Historic American Engineering Record/Historic American Landscapes Survey mitigation documentation. She prepares cultural resources sections for CEQA environmental documents, including infill checklists, initial studies, environmental impact reports, and NEPA environmental documents, including environmental impact statements and environmental assessments. She also specializes in municipal preservation planning, historic preservation ordinance updates, Native American consultation, and provision of Certified Local Government training to interested local governments. She develops Survey 123 and Esri Collector applications for large-scale historic resources surveys, and authors National Register nomination packets. Margo meets the Secretary of the Interior's Professional Qualification Standards for history and architectural history.

Sincerely,

Peter Kloess, PhD
Senior Paleontologist/Principal Investigator

Attachments:

Attachment 1 – Figures

Attachment 2 – Records Search Results

REFERENCES

CGS (California Geological Society). 2002. "California Geomorphic Provinces Note 36." https://www.conservation.ca.gov/cgs/Documents/Publications/CGS-Notes/CGS-Note-36.pdf.

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PBDB (The Paleobiology Database). 2023. Project site search. Electronic database. Accessed October 2023. https://paleobiodb.org/#/.

SDNHM (San Diego Natural History Museum). 2023. Project site search. Paleontology Collections Database. Accessed October 2023. https://www.sdnhm.org/science/paleontology/resources/collection-database.

SVP (Society of Vertebrate Paleontology). 2010. "Standard Procedures for the Assessment and Mitigation of Adverse Impacts to Paleontological Resources." https://vertpaleo.org/wp-content/uploads/2021/01/SVP_Impact_Mitigation_Guidelines.pdf.

UCMP (University of California Museum of Paleontology) Locality Search. 2023. Project site search. Electronic database. Accessed October 2023. https://ucmpdb.berkeley.edu/loc.html.

3100 Zinfandel Drive, Suite 125, Rancho Cordova, CA 95670
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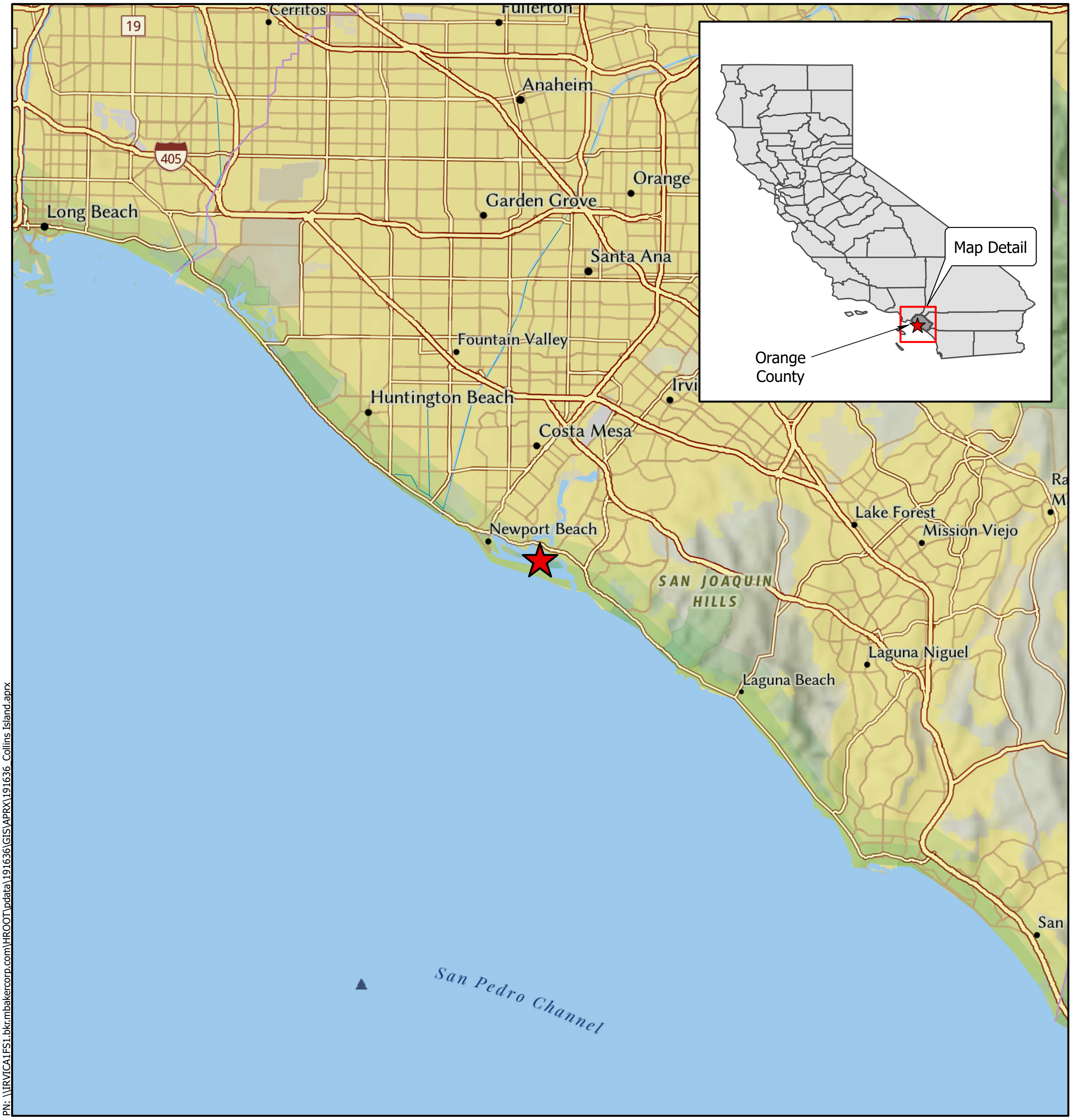
Attachment 1

Figures



Legend

Project Area

Michael Baker INTERNATIONAL

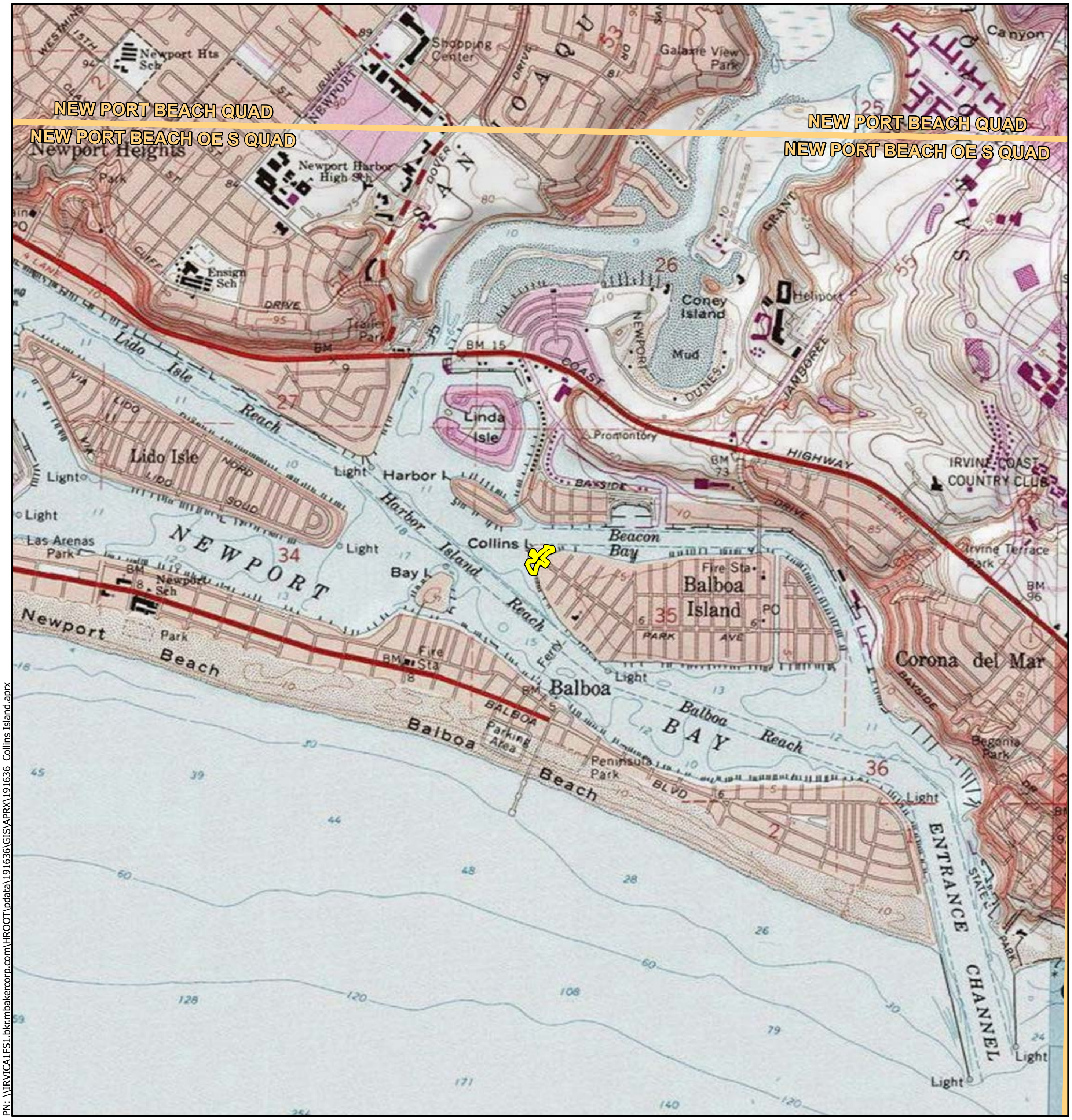
0 2.5 5 Miles

Source: Esri, ArcGIS Online, National Geographic World Map: *Newport Beach, California*

COLLINS ISLAND BRIDGE
NEWPORT BEACH, CA

Regional Vicinity

Figure 1



Legend

Project Area

Michael Baker INTERNATIONAL

0 0.25 0.5 Miles

COLLINS ISLAND BRIDGE
NEWPORT BEACH, CA

Project Vicinity

Source: Esri, ArcGIS Online, Newport Beach OE S USGS 7.5-Minute topographic quadrangle maps: *Newport Beach, California*

Figure 2

PN: \\IRVICA1FS1.bkr.mbakercorp.com\HROOT\pdata\191636\GIS\APRX\191636_Collins_Island.aprx

Legend

Project Area

Michael Baker INTERNATIONAL

0 50 100 Feet

Source: Esri, ArcGIS Online, 2023 Nearmap Imagery: *Newport Beach, California*

COLLINS ISLAND BRIDGE
NEWPORT BEACH, CA

Project Area

Figure 3

Attachment 2

Records Search Results

Natural History Museum
of Los Angeles County
900 Exposition Boulevard
Los Angeles, CA 90007

tel 213.763.DINO
www.nhm.org

Research & Collections

e-mail: **paleorecords@nhm.org**

August 20, 2023

Michael Baker International
Attn: Marc Beherec

re: Paleontological resources for the Collins Bridge Replacement Project

Dear Marc:

I have conducted a thorough search of our paleontology collection records for the locality and specimen data for proposed development at the Collins Bridge Replacement Project area as outlined on the portion of the Newport Beach OE S USGS topographic quadrangle map that you sent to me via e-mail on August 18, 2023. We do not have any fossil localities that lie directly within the proposed project area, but we do have fossil localities nearby from the same sedimentary deposits that occur in the proposed project area, either at the surface or at depth.

The following table shows the closest known localities in the collection of the Natural History Museum of Los Angeles County (NHMLA).

Locality Number	Location	Formation	Taxa	Depth
LACM VP 5466	Northwest corner of the intersection of Jamboree Road & Pacific Coast Highway; Newport Beach	Palos Verdes Sand	Horse (*Equus*)	Unknown
LACM IP 25839	Promontory Drive, Bayport, Newport Bay	Palos Verdes Sand	Invertebrates (including *Eulithidium, Flossaulax, Ostrea, Donax, Callianax, Mitrella, Chama, Balcis, Strioterebrum, Melampus, Amphissa, Aesopus, Laevicardium, Bulla, Crepidula, Dentalium, Leptopecten, Tellina*)	Unknown
LACM IP 31435, 4760	Newport Bay (more precise location not available)	Palos Verdes Sand	Invertbrates: bivalves (*Leukoma, Ostrea,Crepidula, Saxidomus, Tresus, Leptopecten, Macoma, Lucinisca, Brachidontes, Tellina*) ; gastropods (*Caesia, Pseudomelatoma, Callianax*); barnacle (Balanidae); scaphopod (Dentalium)	Unknown
LACM VP 4254; LACM IP 17103, 17104	Corona del Mar Plaza, Newport Beach	Palos Verdes Sand	Duck (Chendytes); saltwater clam (*Chama, Septifer, Epilucina*), mussel (*Mytilus*), sponge trace (*Entobia*),	Unknown

Locality Number	Location	Formation	Taxa	Depth
			feeding trace (*Oichnus*), oyster (*Ostrea*), turban snail (*Megastraea*), limpet (*Lottia*)	
LACM VP 3408	West of the intersection of Ford Road and Jamboree Road	Fernando Formation	Shark (Charcharodon, Carcharocles; *Alopias*); fragmentary marine mammal bones	unknown
LACM VP 3407, 4426; LACM IP 71, 5627	Top of roadcut East side of McArthur Blvd. approx. 1/2 mile S. of Bonita Canyon intersection.	Palos Verdes Sand	Mammoth (*Mammuthus*); and uncatalogued birds, fish, mammals, and invertebrates	Surface

VP, Vertebrate Paleontology; IP, Invertebrate Paleontology; bgs, below ground surface

This records search covers only the records of the NHMLA. It is not intended as a paleontological assessment of the project area for the purposes of CEQA or NEPA. Potentially fossil-bearing units are present in the project area, either at the surface or in the subsurface. As such, NHMLA recommends that a full paleontological assessment of the project area be conducted by a paleontologist meeting Bureau of Land Management or Society of Vertebrate Paleontology standards.

Sincerely,

Alyssa Bell, Ph.D.
Natural History Museum of Los Angeles County

enclosure: invoice