Appendix H-2

Preliminary Foundation Report

Earth Mechanics, Inc.
Geotechnical & Earthquake Engineering

March 14, 2022

EMI Project 20-156

Moffatt & Nichol
1660 Hotel Circle North, Suite 500
San Diego, CA 92123

Attention: Mr. Jared Cole, PE

Copy: Ms. Alex Ford, PE

Subject: **Preliminary Foundation Report**
Pier View Way Bridge and Lifeguard HQ Project
City of Oceanside, California

Dear Mr. Cole:

Attached is our Preliminary Foundation Report (PFR) for the proposed Pier View Way Bridge and Lifeguard Headquarters project. This report is prepared to support the preliminary design phase of the project.

The recommendations and conclusions provided in this report are based on very limited available subsurface soil information. These conclusions and recommendations are considered preliminary and should be confirmed following completion of a site-specific geotechnical field investigation, laboratory testing program, and additional engineering analyses.

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.

Jody Castle, CEG 2592
Senior Geologist

Eric Brown, GE 2806
Principal Engineer

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PRELIMINARY FOUNDATION REPORT
PIER VIEW WAY BRIDGE AND LIFEGUARD HQ PROJECT
CITY OF OCEANSIDE, CALIFORNIA

Prepared for:

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EMI Project 20-156

March 8, 2022

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1.0 INTRODUCTION

This Preliminary Foundation Report (PFR) has been prepared to provide the preliminary geotechnical information to assist the structural designers in the planning phase of the Pier View Way Bridge and Lifeguard Headquarters project. The contents of this PFR generally follow the reference titled "Foundation Reports for Bridges" (Caltrans, 2021). In addition to the information provided for the Pier View Way Bridge, which will be designed according to Caltrans guidelines, preliminary geotechnical recommendations are provided for the Lifeguard Headquarters that will be designed according to California Building Code (CBC, 2019) guidelines.

The preliminary recommendations provided in this report are based on the very limited subsurface information contained in the reference documents included with the project Request for Proposals. A site-specific geotechnical investigation will be performed for the subject improvements during the final design phase; therefore, the following preliminary recommendations require verification when additional site-specific information becomes available.

The geotechnical scope of work included: (1) reviewing available geotechnical/geologic information including reports, published geologic maps, and seismic hazard reports, (2) reviewing as-built plans of the existing structure, (3) assessing the foundation types for the existing or proposed structure, (4) engineering analyses; and (5) preparation of this PFR. The geotechnical and geologic references reviewed for this project are listed in the references section of this report.

2.0 PROJECT DESCRIPTION

The project site is located in the City of Oceanside where Pier View Way spans between North Pacific Street and the Oceanside Municipal Pier. The Oceanside Municipal Pier consists of two distinct elements, the concrete Pier View Way Bridge constructed in the mid-1920's and the timber pier constructed in the late 1980's.

The Pier View Way Bridge (Bridge) is constructed of reinforced concrete with two lanes divided by a vehicle access ramp. Exterior stairways are located on either side of the Bridge at the eastern end to provide pedestrian access from Pier View Way to The Strand that passes beneath the Bridge. The structure comprises a concrete slab spanning between longitudinal beams that frame into bridge piers consisting of concrete columns and transverse beams. The Bridge has 15 spans with the three westernmost supports founded on 16 ft long timber piles and the eastern supports founded on spread footings. The lifeguard headquarters was incorporated underneath the western end (between the two westernmost spans) of the bridge in the mid-1980s.

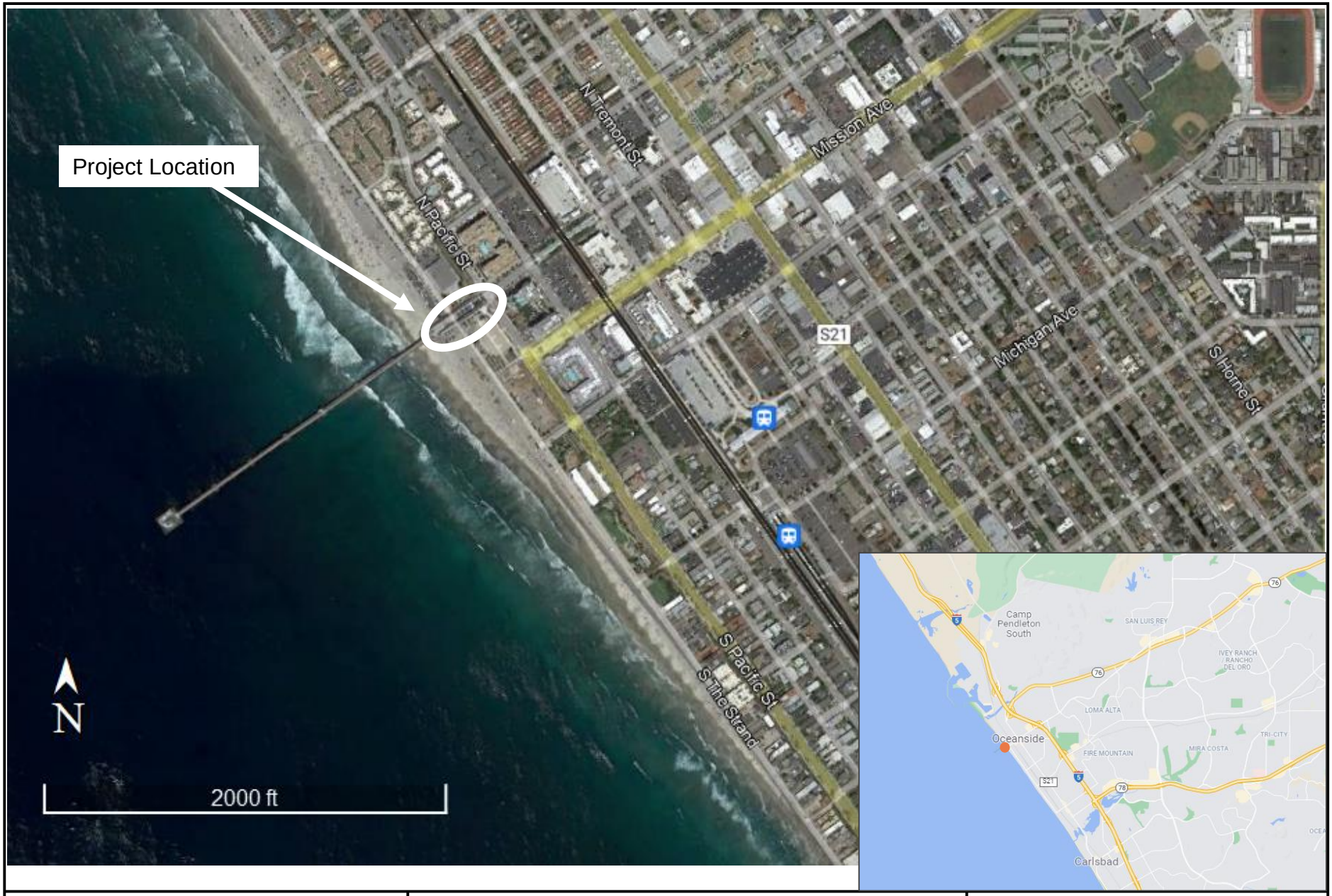
The timber pier (Pier) comprises a timber deck supported by timber stringers and pile caps that are in turn supported by timber piles. The piles are supported laterally with a steel pipe bracing system. The Pier is in good condition, except for the steel pipe bracing and certain connection hardware elements that are in poor condition, primarily due to corrosion of the steel material. The Pier remains capable of sustaining the original vertical design loads and is not part of the subject project improvements. Copies of relevant pages of the Bridge plans are included in Appendix A.

The Bridge was rated as being in "fair" condition in 2007, primarily because of concrete degradation associated with corrosion of reinforcing steel, a common phenomenon with reinforced concrete structures in the marine environment. The bridge has been assigned a "poor" condition

rating as of 2018 due to an additional 10 years of service and observations made during the 2016 to 2018 concrete maintenance work.

The existing Bridge and Lifeguard Headquarters will be reconstructed or restored due to the poor condition rating. The structure(s) will be located along the same alignment as the existing structure and within the same structure footprint. The Pier will remain in its current condition with minor serviceability improvements. The location of the project site is shown in Figure 1.

Vertical control of the original as-built information for the structure is based on the National Geodetic Vertical Datum of 1929 (NGVD 29). A vertical adjustment will be provided by the project surveyor to convert the original datum (NGVD 29) of the as-built plans to the current datum (NAVD 88) during the PS&E phase of the project.



Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

Pier View Way Bridge and Lifeguard HQ Project

Project No. 20-156

Date: March 2022

Site Location Map

Figure 1

3.0 EXCEPTIONS TO POLICIES AND PROCEDURES

From a geotechnical standpoint, there are no known geotechnical conditions that would cause deviation from Caltrans policies or procedures that require an exception. If the site-specific geotechnical investigation reveals that site soils are classified S2 per Caltrans SDC (2019c), then a fixed pile head condition would be required. Lifeguard Headquarters will be designed according to the California Building Code (2019).

4.0 GEOTECHNICAL INVESTIGATION

No site-specific geotechnical field investigation was performed specifically in preparation of this PFR. There is no geotechnical information included on the structure plans for the Bridge. TerraCosta Consulting Group (TerraCosta) provided geotechnical input (TerraCosta, 2007a) for a Design Wave Analysis (Terracosta, 2007b) in 2007. The geotechnical study performed by TerraCosta indicates several sources of information were reviewed for the study, but none were included as attachments to the report.

5.0 LABORATORY TESTING PROGRAM

No site-specific laboratory testing was performed specifically in preparation for this PFR. No available geotechnical laboratory test results from previously conducted investigations were available for review as part of the preparation of the subject report.

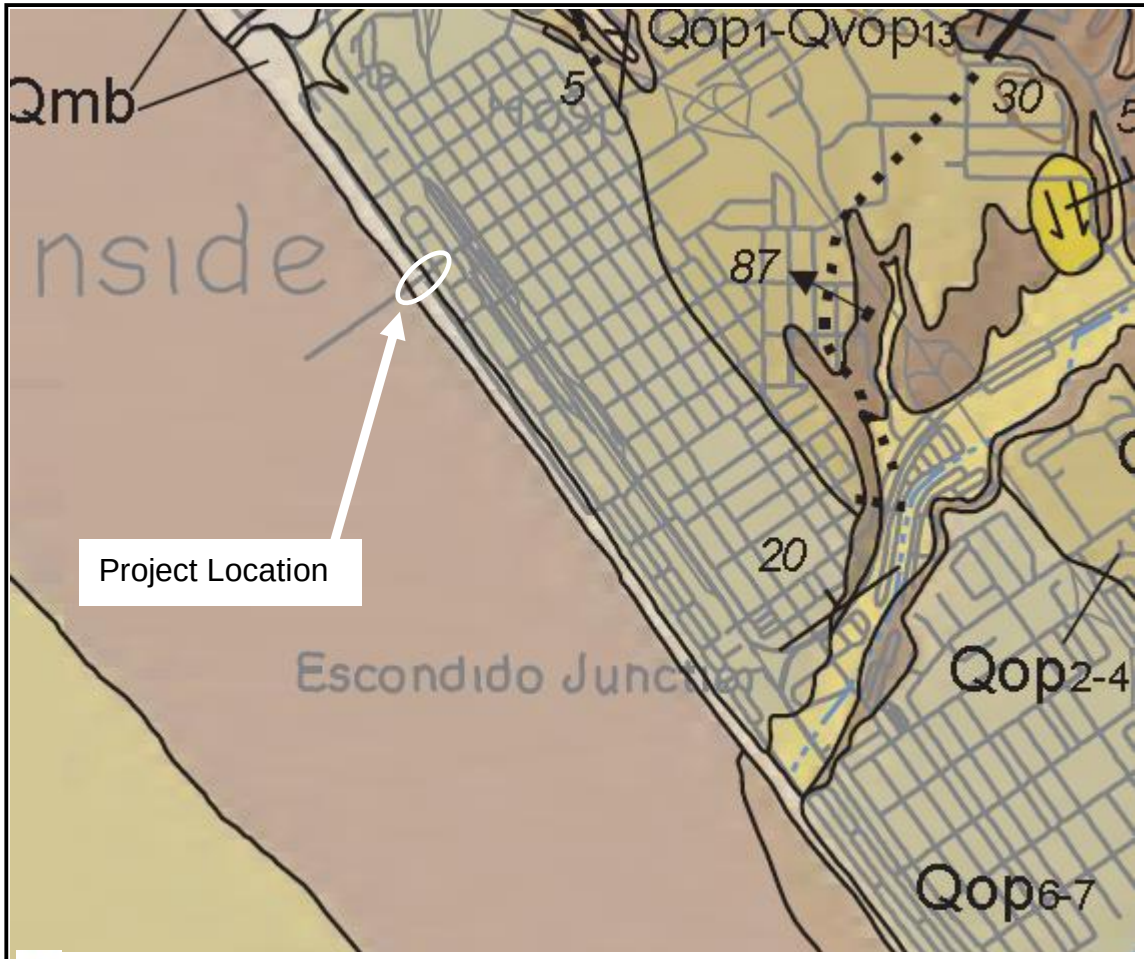
6.0 GEOTECHNICAL CONDITIONS

6.1 Geology

The project area is located in the western portion of the Peninsular Ranges physiographic province, in the Coastal Plain subregion. The Coastal Plain region, ranging from approximately 1 to 12 miles wide, is bounded by the Pacific Ocean to the west, and the Peninsular Ranges to the east. It is characterized by broad, planar mesas gently sloping to the west, incised by deep canyons. The Peninsular Ranges are a group of northwest-southeast trending mountains and valleys between the San Andreas fault on the east and the offshore area called the Continental Borderland. Bedrock in the Peninsular Ranges is predominantly composed of Mesozoic-age granitics. The region surrounding Oceanside, including the offshore Continental Borderland area, is transected by a series of long, mostly northwest-trending, strike-slip fault systems.

Published geological mapping (Figure 2) indicates that surficial soils at the project site consist of consolidated old paralic deposits (Qop). From the coastal bluffs west to the ocean, surficial soils consist of unconsolidated beach sands (Qmb) (Kennedy et al., 2007).

The project alignment, while not crossed by any known faults or projected fault trends, is located in a seismically active region and is subject to shaking from both local and distant earthquakes. Active regional faults closest to the project site include the offshore section of the Newport-Inglewood fault and the Oceanside section of the Rose Canyon fault. Generally, these sections are considered to be one continuous fault zone (USGS, 2021). The north-south trending Newport-Inglewood-Rose Canyon fault zone is located approximately 2.4 miles east of the project. A dextral strike-slip fault, the fault is considered to be capable of producing a magnitude 6.9 earthquake.



N

SCALE

0 1.0 2.0 miles

Map Source:
Kennedy, M.P., Tan, S.S., Bovard, K.R., Alvarez, R.M., Watson, M.J., and Gutierrez, C.I., 2007, Geologic map of the Oceanside 30x60-minute quadrangle, California: California Geological Survey, Regional Geologic Map No. 2, scale 1:100,000

GEOLOGIC SYMBOLS

not all symbols shown on each map

FORMATION CONTACT *dashed where inferred or indefinite dotted where concealed*

MEMBER CONTACT *between units of a formation* Prominent bed

CONTACT BETWEEN SURFICIAL SEDIMENTS *located only approximately in places*

FAULT: *Dashed where indefinite or inferred, dotted where concealed, queried where existence is doubtful. Parallel arrows indicate inferred relative lateral movement. Relative vertical movement is shown by U/D (U=upthrown side, D=downthrown side). Short arrow indicates dip of fault plane. Sawteeth are on upper plate of low angle thrust fault.*

FOLDS: overturned — ANTICLINE — SYNCLINE

arrow on axial trace of fold indicates direction of plunge; dotted where concealed by surficial sediments

Strike and dip of sedimentary rocks: inclined; inclined (approximate); overturned; horizontal; vertical

Strike and dip of metamorphic or igneous rock foliation or flow banding or compositional layers: inclined; inclined (approximate); vertical; overturned

OTHER SYMBOLS: Direction of landslide movement; outline of water bodies shown on map; water well; oil well; springs

Qya	Young alluvial flood-plain deposits (Holocene and late Pleistocene)
Qls	Landslide deposits, undivided (Holocene and Pleistocene)
Qmb	Marine beach deposits (late Holocene)
Qop	Old paralic deposits, undivided (late to middle Pleistocene)
Qop_7	Unit 7
Qop_6	Unit 6
Qop_4	Unit 4
Qop_3	Unit 3
Qop_1	Unit 1
Qop_{6-7}	Units 6-7
Qop_{4-6}	Units 4-6
Qop_{2-4}	Units 2-4
Qop_{1-2}	Units 1-2
Qop_{7-8}	Units 7-8
Qop_{2-6}	Units 2-6
$Qop_{1-Qvop_{13}}$	Units Qop_1 -$Qvop_{13}$
Tsa	Santiago Formation (middle Eocene)

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

Pier View Way Bridge and Lifeguard HQ Project

Project No. 20-156 | Date: March 2022

Regional Geologic Map

Figure 2

Perhaps the most significant recent earthquake in the region was the 1986 event which occurred offshore from Oceanside. This earthquake had a moment magnitude (M_W) of 5.8. The earthquake caused damage throughout San Diego County. Damage consisted mainly of broken plate-glass windows, cracked walls, and plaster, and broken chimneys.

6.2 Surface Conditions

The project site lies along the coastal shoreline and is bounded by the Oceanside Pier to the west and North Pacific Street to the east. Grade below the majority of the Bridge is generally flat with the western supports located right at the shoreline being protected by a breakwater. The intermediate supports span over The Strand, a local street that provides pedestrian and vehicle access to the beach as well as the lifeguard headquarters, community center, retail shops and an outdoor amphitheater that surround the Bridge. At the eastern end of the Bridge, grade rises about 30 ft via the bluffs at the roadway edge of North Pacific Street where the deck of the Bridge is accessed. The majority of the area beneath the Bridge is covered with hardscape or pavement with the exception of a few areas with grass landscaping and the banks of the bluffs below North Pacific Street.

6.3 Subsurface Conditions

Based on existing geologic mapping (Kennedy et al., 2007), the marine beach deposits (Qmb) supporting the pier and the western end of the Bridge are unconsolidated, loose to medium dense, fine to medium grained sands. The deeper beach deposits were derived from sediments being carried down the San Luis Rey River. Construction of Lake Henshaw in 1922 reduced the sediment-carrying capacity of the San Luis Rey River and thereby diminished the natural supply of beach material to Oceanside beaches. During the late 1880s the beach off Oceanside was approximately 300 feet wide (USCGS 1889). By 1925 the usable beach was quite narrow, and the beach width could be correlated with the stream runoff rate from the river (Hales 1978). The last storm-flood year to contribute appreciable amounts of sediment to southern California beaches was 1938 (Troxell et al. 1942).

The coastal bluffs above the beach are composed of older Quaternary paralic deposits (Qop) consisting of interfingered strandline, beach, estuarine and colluvial deposits of siltstone, sandstone, and conglomerate. The soils are expected to be moderately well consolidated and dense to very dense. The bluffs are historically capable of standing near vertical due to cementation within the siltstone, sandstone, and conglomerate layers. Prior to development surrounding the pier, the greatest detriment to bluff stability prior to the narrowing of the beach in the mid 1920's, was subaerial erosion. After the narrowing of the beach, between late December 1940 and early January 1941, a series of storms originating in the Aleutian Islands moved into southern California and considerably damaged beachfront and bluff-top property (Kuhn and Shepard 1979a). The project area has since been covered with hardscape and landscape associated with the pier construction and surrounding development and is now largely resistant to subaerial and storm related erosion.

The bluffs have not historically been susceptible to landslides and are expected to be globally stable even during the design seismic event. The bluffs have been modified as part of the development surrounding the pier and are suitable for support of habitable structures on spread

footings without requiring extensive foundations. The bluff materials are expected to be easily excavated with conventional earth moving equipment.

Lateral soil springs (P-Y curves) are provided in the TerraCosta (2007) report to support the Wave Analysis and as stated in the report "The site soils appear to be generally comprised of sands throughout the depth of pile embedment. We have assumed that these sands have a relative density of 50 percent. In addition, we understand that the existing pier piles consist of 18-inch timber piles that have a minimum depth of embedment of 20 ft below the bottom of the design scour depth."

The above description is consistent with the available information for regional geologic maps (see Section 6.0). The preliminary recommendations provided in this report assume that the near-surface profile below the Bridge consist of saturated, medium dense sands within the depth of penetration of the existing structure foundations.

7.0 GROUNDWATER

Due to the site's proximity to the ocean, groundwater will be influenced by tidal fluctuations. For preliminary design purposes, the groundwater is placed at the high tide elevation. Based on current National Oceanic and Atmospheric Administration (NOAA) tide chart, high tide for San Diego (Station ID 9410170) is approximately +7 ft. (NAVD88). Sea level rise may also be included in the project requirements which would likely raise the project design groundwater elevation for final design and would be determined as part of the environmental study.

8.0 AS-BUILT DATA

As described in Section 2, the bridge has 15 spans with the three westernmost supports founded on 16 ft long timber piles and the eastern supports founded on spread footings. There are no pile design loads or design bearing pressures for the footings included on the structure plans. The lifeguard headquarters was incorporated underneath the western end (between the two westernmost spans) of the bridge in the mid-1980s. There are no plans available for the lifeguard headquarters and it is assumed to be supported on a slab on grade.

9.0 SCOUR DATA

The entire footprint of the project area below the bridge is covered with concrete hardscape. Scour is not considered a design issue for the subject structures.

10.0 CORROSION EVALUATION

No soil corrosion lab tests at the site are currently available. Therefore, corrosion potential of on-site soils is not known. Due to the site's proximity to the ocean, site soils are anticipated to contain elevated chloride levels and be considered corrosive to concrete and steel. If site soils are found to be corrosive, possible mitigation measures include increased cover for reinforcing steel, epoxy coated steel, corrosion resistant cement (in case of concrete piles and shallow foundations), and sacrificial steel can be provided for steel surfaces in contact with site soils.

11.0 SEISMIC INFORMATION

11.1 Ground Motion Hazard

The site is in seismically active southern California and is subject to shaking from both local and distant earthquakes. Large events on the nearby Newport-Inglewood fault control seismic design of the project.

Table 1. Fault Data

Fault	Fault ID	Fault Type(1)	Maximum Earthquake Magnitude	Approx. Distance from Site to Fault (miles)	Surface Fault/Blind Fault
Newport-Inglewood (offshore)	381	SS	6.9	2.4	Surface
Rose Canyon (Oceanside section)	396	SS	6.8	4.6	Surface

Note:
(1). SS = Strike Slip.

11.2 Other Seismic Hazards

11.2.1 Pier View Way Bridge

The preliminary ARS curve for the 975-year return event was determined using the Caltrans ARS Online V3.0.2 (2019a). The small-strain shear wave velocity for the upper 100 feet (Vs_{30}). Vs_{30} was estimated based on the near surface profile described in Section 6 and our review of regional geologic maps. The Vs_{30} and design response spectrum will be updated once the field investigation is completed after environmental clearance has been secured. Key information such as the site coordinates and assumed shear wave velocity for determining the preliminary ARS curve are listed in Table 2.

Table 2. Preliminary ARS Curve

Latitude	Longitude	Shear Wave Velocity, V_{s30}	Design Earthquake Return Period	Peak Ground Acceleration (PGA)	Mean Magnitude	Mean Site-Source Distance (km)
33.1944°	-117.3842°	820 ft/sec (250 m/sec)	975	0.4	6.64	33.5

The preliminary ARS curve is presented in Figure 3. Near-fault factors were not applied per Caltrans SDC V2.0 (2019c) guidelines based on the mean site to source distance from the deaggregation being greater than 25 km. The peak ground acceleration (PGA), mean magnitude (for PGA) and mean site-source distance (for Sa at 1 second) is listed in Table 2. The soil class at the proposed structure location is estimated to be S2 per Caltrans SDC V2.0 (2019c).

The design ARS curve, magnitude, and site classification will be updated during the final design phase following the field investigation based on a Vs_{30} determined using SPT correlations obtained from site-specific soil borings.

11.2.2 Lifeguard Headquarters

For the purposes of preliminary design, near surface site soils are assumed to be liquefiable. Due to the presence of potentially liquefiable materials, the site is classified as Class F for seismic evaluations of structures per California Building Code (CBC, 2019) and ASCE 7-16 (ASCE, 2016). Therefore, site-specific design Acceleration Response Spectrum (ARS) should be developed per ASCE 7-16 guidelines (ASCE, 2016). The site-specific ARS curve was developed using the USGS Unified Hazard Tool with the Dynamic: Conterminous U.S. 2014 (V4.2.0) edition conservatively using Sight Class D (V_{s30} = 259 m/s). Table 3 below provides the recommended design earthquake parameters per ASCE 7-16 Section 21.4.

Table 3. Input Seismic Parameters

Parameter	Value per CBC 2019/ASCE 7-16
S_s	0.993g
S_1	0.364g
F_a	1.103
F_v	1.936 [1]
S_{DS}	0.730g
S_{D1}	0.470g [1]
T_L	8 seconds
PGA_M	0.506g
Note: (1) The requirements outlined in ASCE 7-16 – Section 11.4.8 – Exception No. 2 should be satisfied. The recommended F_v should be used only for calculation of T_s per ASCE 7-16 Standard.	

11.3 Liquefaction Potential

In the absence of site-specific geotechnical information, it is assumed that the near surface deposits below the majority of the pier bridge are unconsolidated beach sands that were subject to repeated cycles of scour and deposition by wave action prior to construction of the Bridge. These near surface soils have sufficient bearing capacity to support the existing structure with no signs of settlement or distress; however, it is expected that during seismic shaking the unconsolidated nature of the sandy soils in conjunction with the shallow groundwater will result in a high potential for liquefaction at the site. The soils that support the bluff on the east end of the Bridge have proven over time to be resistant to erosion and are composed of old paralic (interfingered continental and marine) deposits that are expected have a low potential for liquefaction. This conclusion will be confirmed using additional site-specific soil borings and groundwater data to be obtained later during the PS&E phase.

11.4 Seismically-Induced Settlement

Since the liquefaction potential is expected to be high below the majority of the project site, the magnitude of liquefaction-induced (seismic) settlement of onsite soils is anticipated to be on the order of several inches to a foot. However, this conclusion will need to be confirmed using site specific soil borings to be performed later during the PS&E phase.

11.5 Lateral Spreading Potential

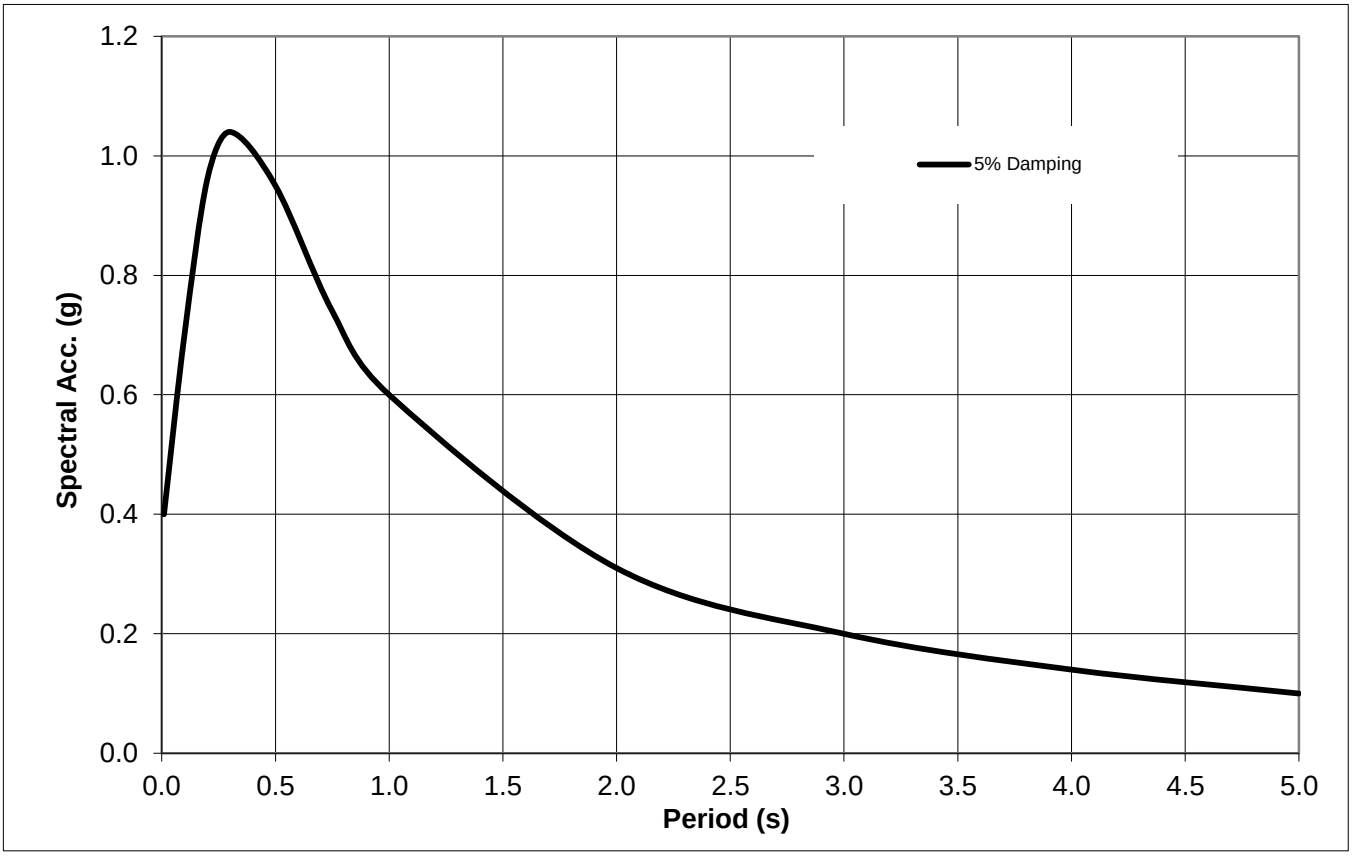
The western portion of the project supporting the Bridge with a high potential for liquefaction is generally flat. There are no submerged slopes within the project area. Therefore, the potential for lateral spreading at the site is considered low.

11.6 Seismic Slope Instability

Since liquefaction potential of the soils supporting the bluff on the eastern end of the project is expected to be low they are not expected to lose strength during the design seismic event. Seismic slope stability is not considered a design issue. Slope stability will be assessed during the PS&E phase using final cross-sections when they become available.

11.7 Surface Fault Rupture

The proposed structure does not fall within an Alquist-Priolo Earthquake Fault Zone or within 1,000 ft of an unzoned fault that is Holocene or younger in age; therefore, in accordance with Caltrans Memo To Designers 20-10 (Caltrans, 2013), further fault studies will not be necessary.



Preliminary ARS Curve

Shear Wave Velocity (V_{s30}) = 250 m/s

Lattitude: 33.1944° Longitude: -117.3842°

5% Damping

ARS Online Version 3.02

Spectral Coordinates:

Period (sec)	**Acc.** (g)
0.01	0.40
0.10	0.70
0.20	0.96
0.30	1.04
0.50	0.95
0.75	0.74
1.00	0.60
2.00	0.31
3.00	0.20
4.00	0.14
5.00	0.10

12.0 GEOTECHNICAL RECOMMENDATIONS

As described in Section 2, the Bridge and Lifeguard Headquarters will be reconstructed or restored with the existing timber Pier remaining as is. The existing structure is supported on spread footings that appear to have performed adequately over time as there are no signs of settlement or distress of the existing super structure. Spread footings are feasible for support for the eastern abutment in the bluffs composed of more consolidated older Quaternary paralic deposits (Qop). However, based on the anticipated high potential for liquefaction beneath the majority of the structure, spread footings are not considered suitable for the remaining supports.

Site soils are expected to be conducive to both driven and drilled deep foundations. Due to the presence of shallow groundwater, "wet" construction methods will be required for Cast-In-Drilled-Hole (CIDH) piles. Both driven steel (pipe piles and HP piles) and driven concrete piles are considered feasible. The older alluvium deposits that comprise the bluffs are anticipated beneath the near-surface unconsolidated beach sands but at a currently unknown depth. Difficult driving conditions for displacement piles is anticipated when the piles encounter the dense, deeper deposits. Depending on the depth at which they are encountered, driven concrete piles may not be preferable due to the potential for early refusal before sufficient embedment is achieved for lateral fixity. That potential cannot be evaluated until a site-specific soil investigation is conducted.

The lifeguard headquarters is anticipated to be a lightly loaded structure capable of being supported on spread footings or a mat foundation. Near-surface site soils above the water table are expected to be predominantly silty sand, sand, and sand with silt, and therefore, they are expected to be non-expansive.

12.1 Shallow Foundations

Bearing Capacity. In the absence of site-specific subsurface information, preliminary design of the foundation for the lifeguard headquarters can be based on presumptive values in Section 1806 of the CBC (2019). A static net allowable bearing capacity of 2,000 psf can be used for mat type foundation with a minimum embedment depth of 12 inches. The allowable bearing capacity may be increased by one-third when considering transient load combinations such as wind and earthquake loading.

Lateral Resistance: The lateral sliding resistance is offered by the friction at the bottom and sides of the footing. We recommend an ultimate frictional coefficient of 0.25 for footings poured against native soil per the CBC (2019).

Lateral Bearing: Lateral bearing pressure of 150 psf per foot of depth up to 2,250 psf is recommended. The resistance from the top one foot of soil should be neglected unless the surface is paved.

The lateral resistances offered by lateral sliding resistance and lateral bearing may be combined to obtain the total lateral resistance.

Modulus of Subgrade Reaction: Modulus of subgrade reaction (Kv1) is estimated to be 70 pci for a square rigid plate having dimensions of 1 ft x 1 ft. For a square foundation having dimension of B (ft) x B (ft), a value of modulus of subgrade reaction K_{BxB} can obtained using the following

equation: $K_{BxB} = K_{v1} ((B+1)/2B)^2$. For a rectangular foundation having a long dimension of L (ft) and short dimension of B (ft), a value of modulus of subgrade reaction K_{LxB} can obtained using the following equation: $K_{LxB} = K_{BxB} [1+(0.5B/L)]/1.5$.

In order to accommodate the uncertainty in estimating the subgrade reaction modulus, use 1.5 and 0.67 as upper-bound and lower-bound multipliers on base modulus of subgrade reaction.

12.2 Deep Foundations

For preliminary design and cost estimating purposes, preliminary axial capacity curves for 24inch diameter CIDH piles, Caltrans Alt "X" concrete piles and Class 90 Alt "W" (PP 14x0.375) piles are provided in Figure 4. For the CIDH piles, 24inch diameter piles are required to accommodate gamma-gamma inspection tubes and the tremie pipe necessary for concrete placement per Caltrans Memo-To-Designers 3-1 (2014).

The preliminary axial pile capacities for CIDH piles were estimated following the Caltrans Amendments to AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications – Eighth Edition (Caltrans, 2019), which are based on the Reese and O'Neil method outlined in Federal Highway Administration (FHWA) Publications FHWA-IF-99-025. The axial capacities for the CIDH were estimated using the computer program SHAFT (2017). The axial capacities for the driven concrete Alt "X" and driven steel Alt "W" piles were estimated according to the recommendations by Tomlinson (1987), where limited values for determining both skin friction and end bearing for cohesionless soils were considered following the recommendations from API (2000). The Alt "W" piles were assumed to be driven open ended.

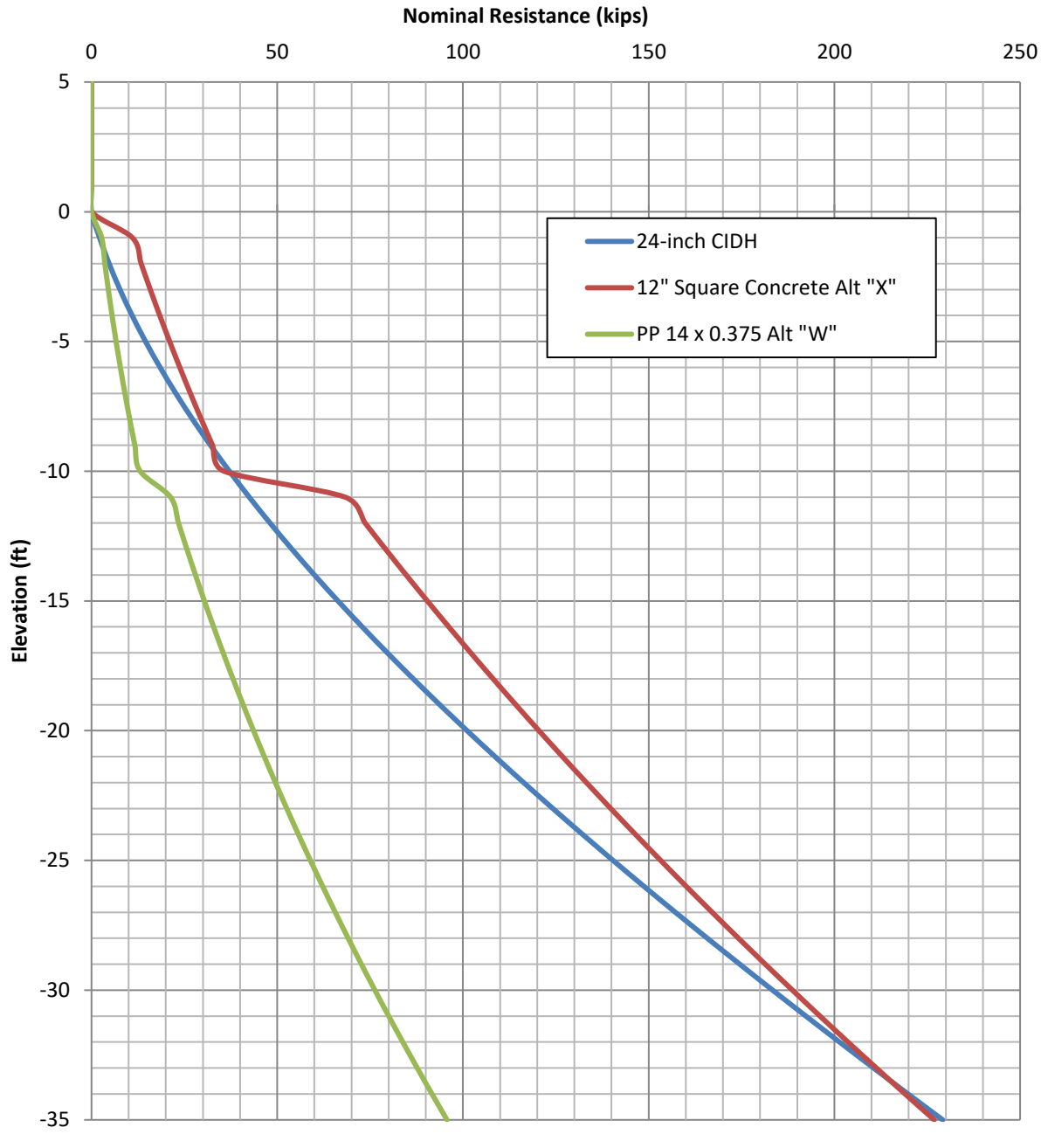
A resistance factor of 0.7 for the strength limit state should be applied to the pile capacities presented in Figure 4 per AASHTO LRFD Bridge Design Specifications (2018). The pile capacities are based on soil resistance only and may be further limited by the pile-head connection details and structural material strength.

13.0 ADDITIONAL FIELD WORK AND LABORATORY TESTING

For final design, EMI intends to perform an exploration program consisting of multiple rotary wash borings. EMI will drill and sample one geotechnical boring through the bluffs at the eastern end of the Bridge. Additional borings will be performed along the length of the Bridge where access permits. Each boring will extend to 15 feet below the bottom of the new proposed foundations.

Samples recovered from the soil borings will be transported to the laboratory for testing. All the soil samples will be visually classified. Additional samples will be collected for moisture content/density tests, sieve analysis, #200 wash, corrosion, Atterberg limit, consolidation, collapse potential, direct shear and unconfined compressive strength testing. Other laboratory tests may be required depending upon the nature of the soils encountered during the investigation.

Nominal Resistance vs Pile Embedment



Notes:
(1) No end bearing assumed for 24" CIDH, both skin friction and end bearing considered for Alt "X" and Alt "W" pile
(2) Assumed cut-off elevation at +0 ft
(3) For strength limit state multiply nominal capacity by a resistance factor of 0.7

Earth Mechanics, Inc. Geotechnical and Earthquake Engineering	Project Number : 21-156	Date: March 2022
	Pier View Way Bridge and Lifeguard HQ Project **Axial Pile Nominal Resistance**	Figure 4

14.0 REFERENCES

American Association of State Highway and Transportation Officials (AASHTO), 2018, AASHTO LRFD Bridge Design Specification, 8th Edition, Washington, DC: AASHTO.

American Petroleum Institute (API), 2000, Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms – Working Stress Design, API RP 2A-WSD, 21st Edition Errata and Supplement.

American Society Civil Engineering (ASCE), 2016, "Minimum Design Loads for Buildings and Other Structures" ASCE/SEI Standard 7-16.

California Building Code (CBC) 2019, "California Building Code," California Code of Regulations, Title 24, Part 2.

California Department of Transportation (Caltrans), 2021, Foundation Reports for Bridges, January.

____, 2019a, ARS Online Web Tool V3.0, http://dap3.dot.ca.gov/ARS_Online/.

——, 2019b, California Amendments to AASHTO LRFD Bridge Design Specifications - Eighth Edition, January.

——, 2019c, Caltrans Seismic Design Criteria, Version 2.0, April.

____, 2018a, Corrosion Guidelines, Version 3.0: Division of Engineering Services, Materials Engineering and Testing Services, Corrosion Branch, March.

____, 2018b, Standard Specifications.

____, 2014, Memo To Designers 3-1, June.

____, 2013, Memo To Designers 20-10, January.

____, 2012, Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendations, November.

Ensoft, 2017, Shaft 2017, A program for analyzing the axial capacity and short-term settlement of drilled shafts under axial loading.

Federal Highway Administration (FHWA), 1999, Drilled Shafts: Construction and Procedures and Design Methods, FHWA-IF-99-2 025.

Hales, L. Z. 1978. Coastal Processes Study of the Oceanside, California Littoral Cell . U. S. Army Corps of Engineers Misc. Paper no. H-78-8. 60 pp., 13 appendixes.

Kennedy, M.P., Tan, S.S., Bovard, K.R., Alvarez, R.M., Watson, M.J., and Gutierrez, C.I., 2007, Geologic map of the Oceanside 30x60-minute quadrangle, California: California Geological Survey, Regional Geologic Map No. 2, scale 1:100,000

Kuhn, G. G. and F. P. Shepard, 1979a . Accelerated Beach-Cliff Erosion Related to Unusual Storms in Southern California. California Geol . 32 (3):58–59.

National Geodetic Survey (NGS), 2020, Data Explorer Web Tool https://www.ngs.noaa.gov/NGSDataExplorer/

Terracosta Consulting Group, 2007a, Design Wave Analysis, Oceanside Pier, Oceanside, California, Project Number 2456, July 9.

Terracosta Consulting Group, 2007b, Phase I Study, Geotechnical Input and Wave Force Estimation, Oceanside Municipal Pier Evaluation, Oceanside, California, Project Number 2456, July 9.

Tomlinson, M. J., 1987, Pile Design and Construction Practice, Scholium International, 3 Sub Edition, March.

Troxell, H. C. et al. 1942. Floods of March 1938 in Southern California . U. S. Geol. Survey Water Supply Paper no. 844. 399 pp.

U.S. Coast and Geodetic Survey, 1889, Descriptive report to accompany original field sheet entitled: Topography: Pacific Coast Southward from San Dieguito Valley, California, Topo. Map no. T-2014. National Ocean Survey Archives, Washington, D. C.

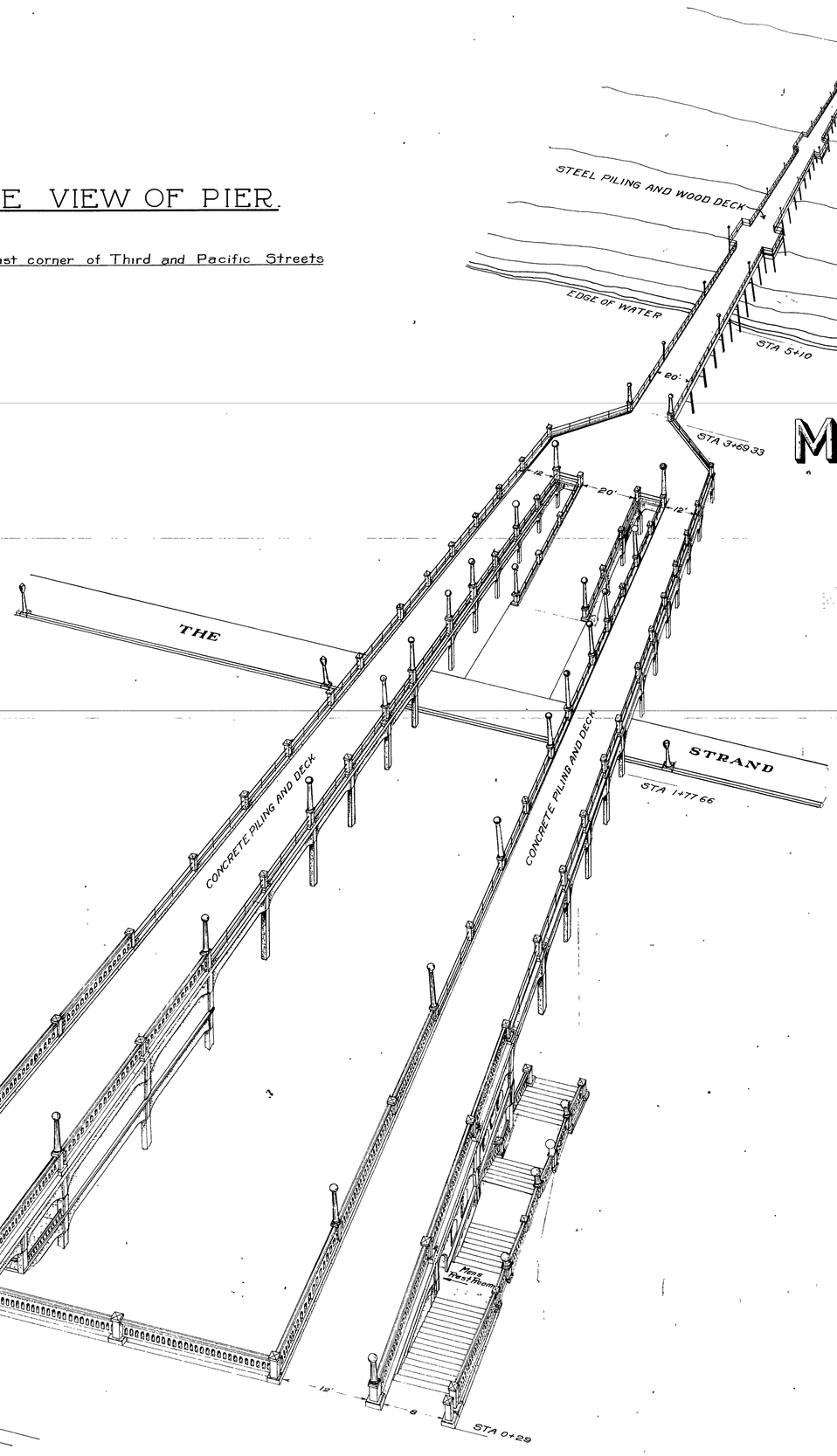
U.S. Geological Survey and California Geologic Survey, Quaternary fault and fold database for the United States, accessed October 8, 2021, at: https://www.usgs.gov/natural-hazards/earthquake-hazards/faults.

APPENDIX A

EXISTING BRIDGE PLANS

PERSPECTIVE VIEW OF PIER.

As seen from over the northeast corner of Third and Pacific Streets



MUNICIPAL PLEASURE PIER.

FOR THE CITY OF

OCEANSIDE, CALIFORNIA.

DESIGNED BY AND BUILT UNDER THE SUPERVISION OF } P.L. Loucks C.E., City Engineer of Oceanside. 1926.

Drawn by - C.S. Seabrook

CONTENTS OF PLANS.	
SHEET	SUBJECT
1.	Title and perspective
2.	General Plan (concrete section)
3.	" " (steel section)
4.	South Rest Room
5.	North Rest Room
6.	Rest Rooms to steel structure
7	Concrete details
8.	" "
9.	" "
10.	Steel details
11.	" "
12.	Pier end

BOARD OF TRUSTEES

E.A. Walsh (PRESIDENT)
Bernard Macdonald
C.D. Merrill
H.D. Brodie
H.F. Crandell
J.H. Landes (CLerk)

3-B

MUNICIPAL PLEASURE PIER.
OCEANSIDE CALIFORNIA.

Sheet 4 of 12 Sheets.

R.L. Loucks - City Engineer

REST ROOM

CONCRETE DECK
Elevation 41.75

BALCONY
Elev 32.38

PLAN.

STA 0+89.06

Elev 41.75

CONCRETE SIDEWALK

WEST CURB LINE OF PACIFIC STREET

PACIFIC STREET

Elev 37.38

STA 0+29.00

STA 0+16

SOUTH WING OF PIER.
PORTION ADJOINING PACIFIC STREET.
REINFORCED CONCRETE CONSTRUCTION.

Scale 1"=4'

Lights and light posts as shown on general plan

SOUTH ELEVATION.

2" GI Pipe Hand Rail.

4" Vents

4" Vents

SIDEWALK
Elev 41.75

CURB

Elev 32.38
BALCONY FLOOR

Top of 12"x12" brace
Elev 23

GROUND PROFILE

22'6" | 22'8 | 20' | 11'0 | 9'0" | 9'0" | 9'0"

MUNICIPAL PLEASURE PIER.
OCEANSIDE CALIFORNIA
Sheet 2 of 12 Sheets
R.L Loucks - City Engineer

GENERAL · PLAN · AND · ELEVATION.

PART - 1

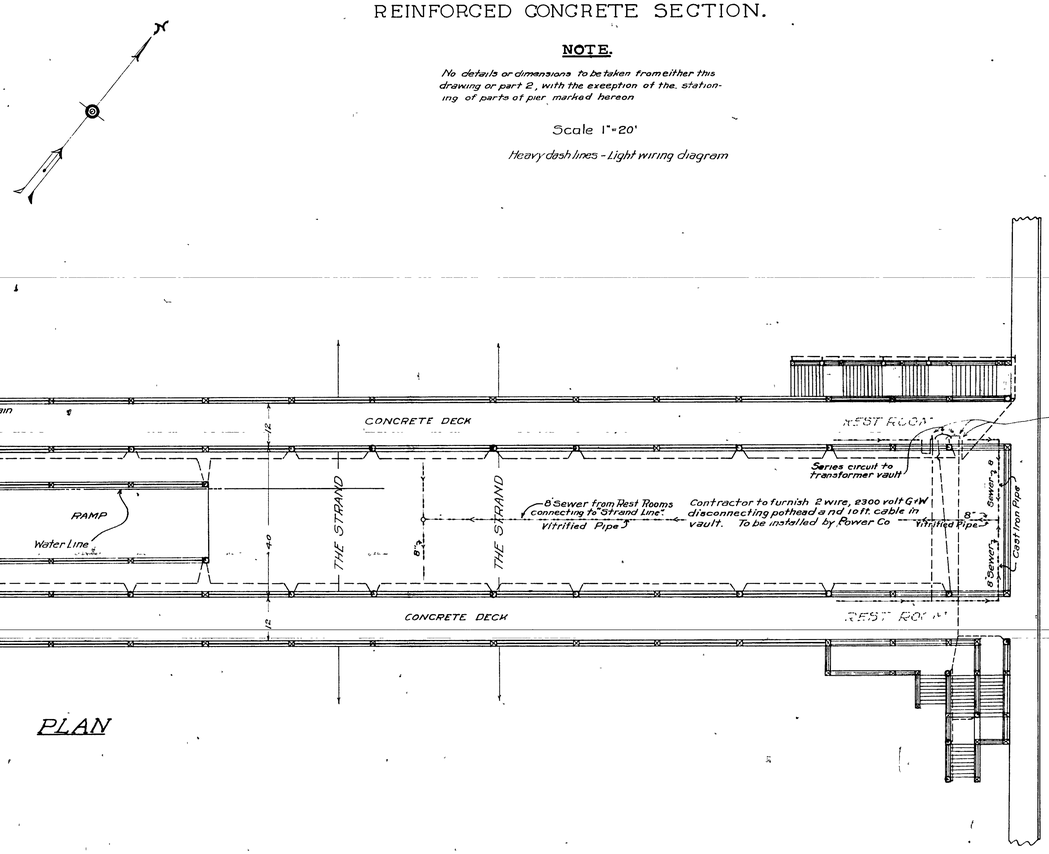
REINFORCED CONCRETE SECTION.

NOTE.

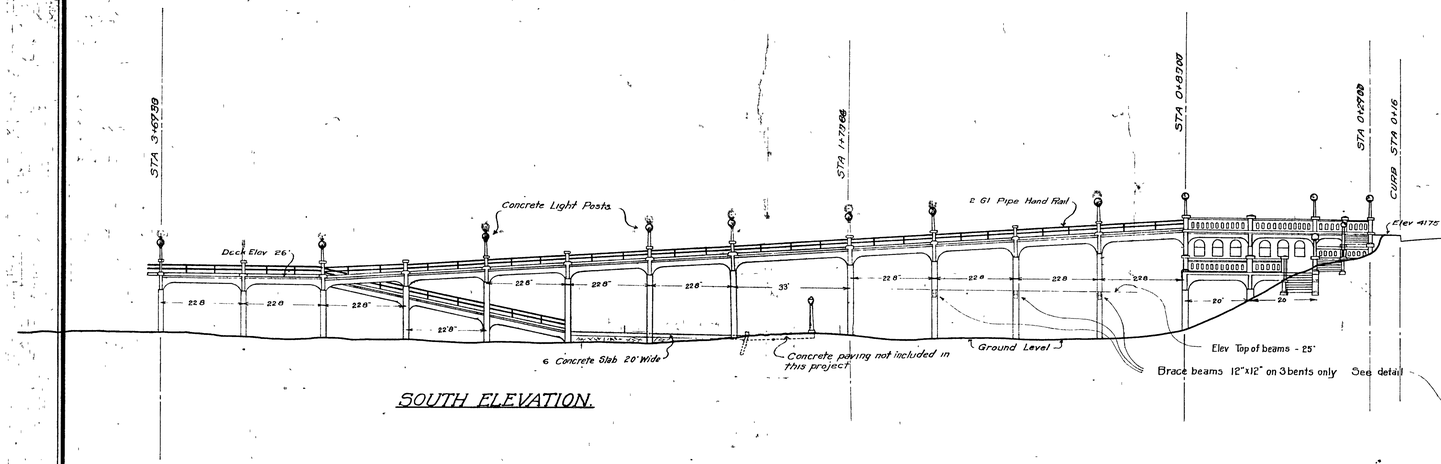
No details or dimensions to be taken from either this drawing or part 2, with the exception of the stationing of parts of pier marked hereon

Scale 1"=20'

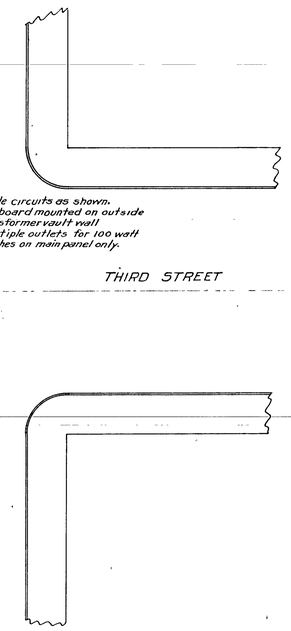
Heavy dash lines - Light wiring diagram

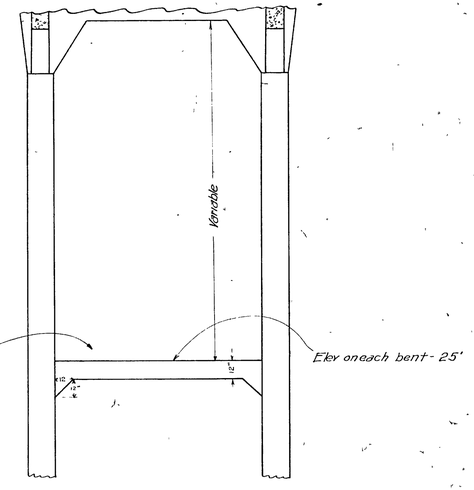


PLAN



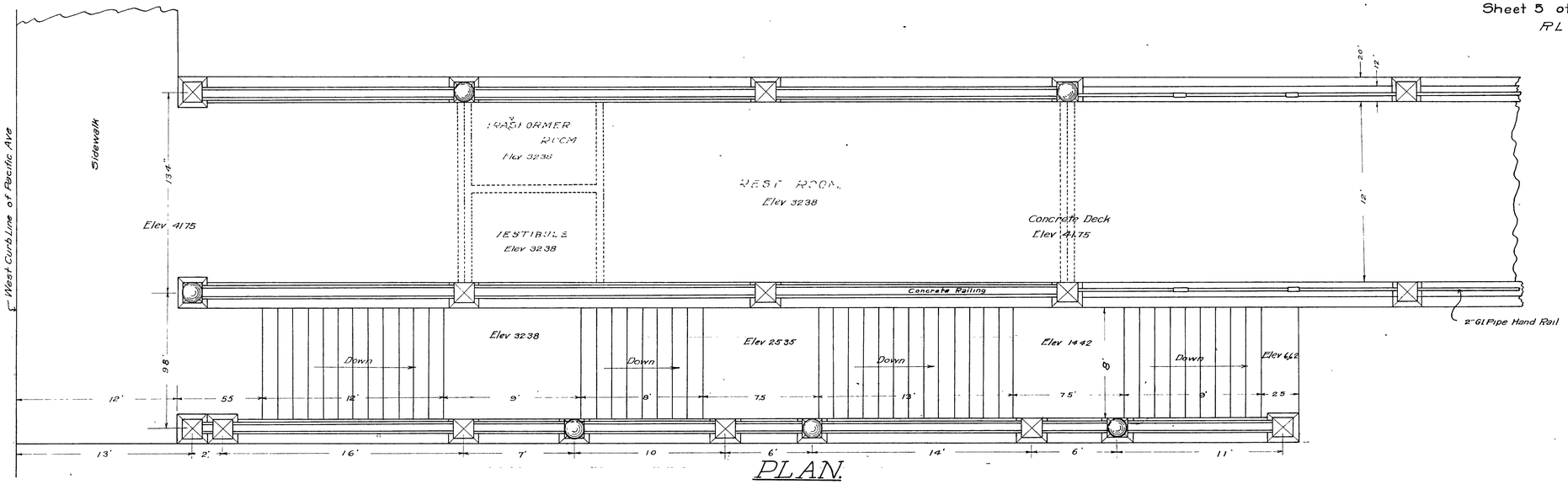
SOUTH ELEVATION.





BRACING BEAM ON 3 BENTS ONLY

MUNICIPAL PLEASURE PIER.
OCEANSIDE, CALIFORNIA.
Sheet 5 of 12 Sheets.
R L Loucks - City Engineer



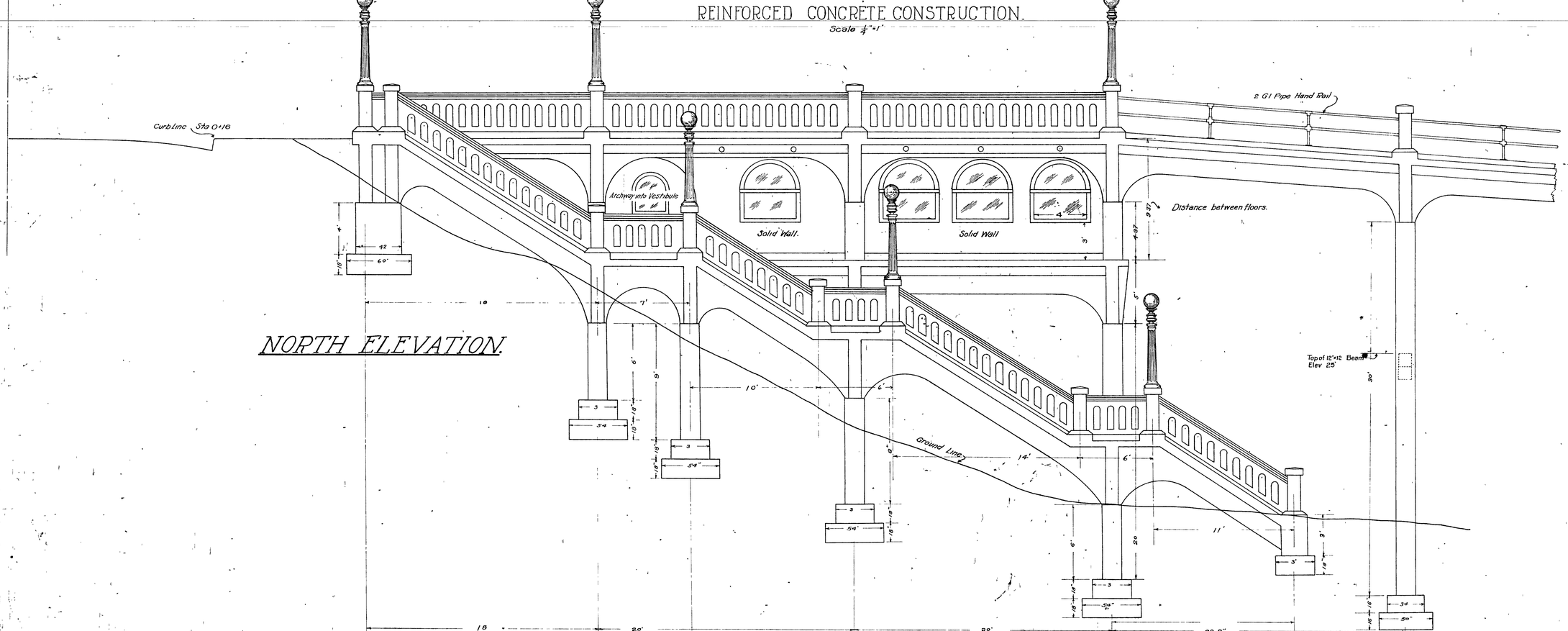
PLAN.

NORTH WING OF PIER.

PORTION ADJOINING PACIFIC AVENUE.

REINFORCED CONCRETE CONSTRUCTION.

Scale 4"=1'



NORTH ELEVATION.

E. & PACIFIC ASE STA 0+00

3-34

MUNICIPAL PLEASURE PIER
OCEANSIDE, CALIFORNIA.

Sheet 6 of 12 Sheets.

R.L. Loucks, City Engineer.

PORTION OF PIER
INCLUDING RAMP

Steel and Wood Construction

Reinforced Concrete Construction

Drinking Fountain

WALK

CONCRETE

DECK

12'

10'

20'

10'

12

CONCRETE

Wood Deck Concrete Deck

Dotted Circles indicating wooden piling beneath footing

67'4"

Elev 26.00

CONCRETE RAMP

TO PIER

TO BEACH

CONCRETE WALK TO STRAND

CONCRETE

DECK

CONCRETE

DECK

PLAN

22'8" 22'8" 22'8" 22'8" 22'8" 22'8" 22'8" 33'

Floor Elev 26.00

Floor Elev 26.00

7'7" 7'7" 7'7" 7'7" 7'7" 8'3" 9'3"

18' 18' 18' 18' 18' 18' 18' 18'

20' 20' 20' 22' 24' 25' 24' 26'

Elev 12.90

6" Concrete slab, 20' long extending from ramp to point on The Strand.

THE STRAND.

GROUND LEVEL

EARTH FILL

These footings are to be 3' wide, 3' deep, and 74' long, being continuous from one side of the pier to the other.

Wood piling, 16' long, one beneath each column as shown

DATUM PLANE MEAN SEA LEVEL

#6 Wire Hoops

5/8 Rods

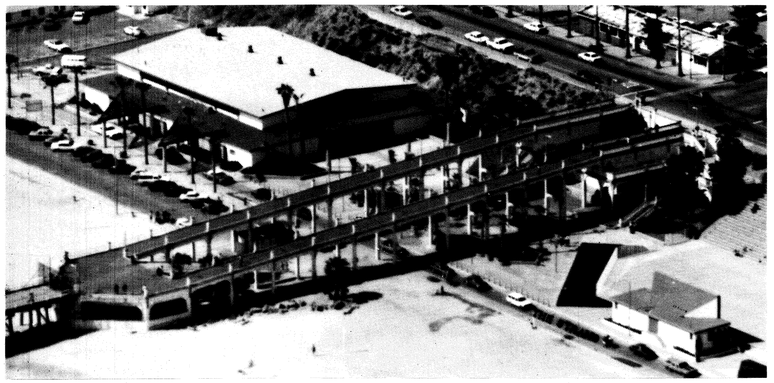
Typical reinforcing to be placed in each footing

SOUTH ELEVATION

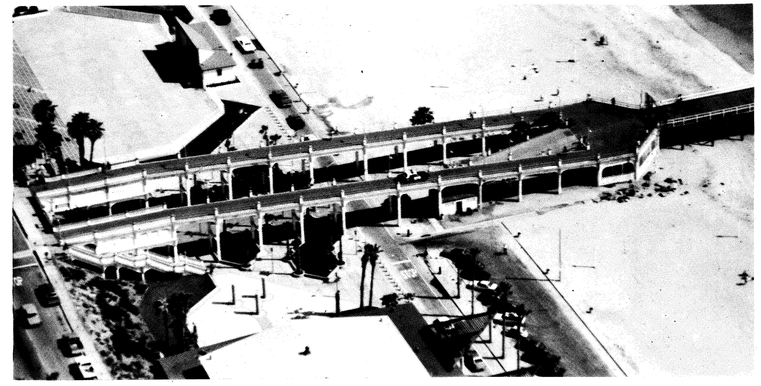
89969

OCEANSIDE PIER RESTORATION

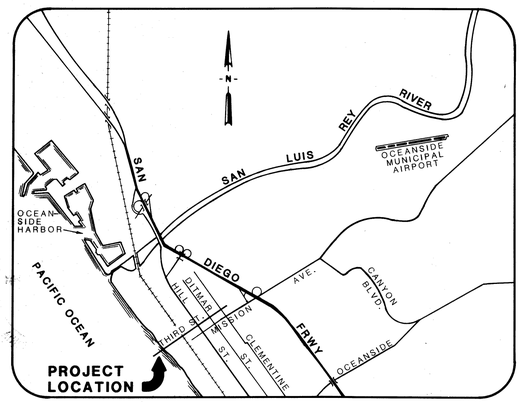
CONCRETE PORTION, INCLUDING LIFEGUARD HEADQUARTERS



LOOKING NORTH



LOOKING SOUTH



LOCATION MAP

SHEET INDEX

REGISTERED PROFESSIONAL ENGINEER
JAMES M. ROSS
20883
CIVIL
STATE OF CALIFORNIA

NOTE:
PHOTOS ON THIS SHEET WERE TAKEN 5-8-85

APPROVED CHANGES

No.	Description	Approved By	Date

BENCH MARK
Description:
Location:
Record From:
Elev: Datum:

SHEET R-1 | CITY OF OCEANSIDE ENGINEERING DEPARTMENT | SHEETS

OCEANSIDE PIER RESTORATION
CONCRETE PORTION/LIFEGUARD HEADQUARTERS

TITLE SHEET

Approved
City Engineer

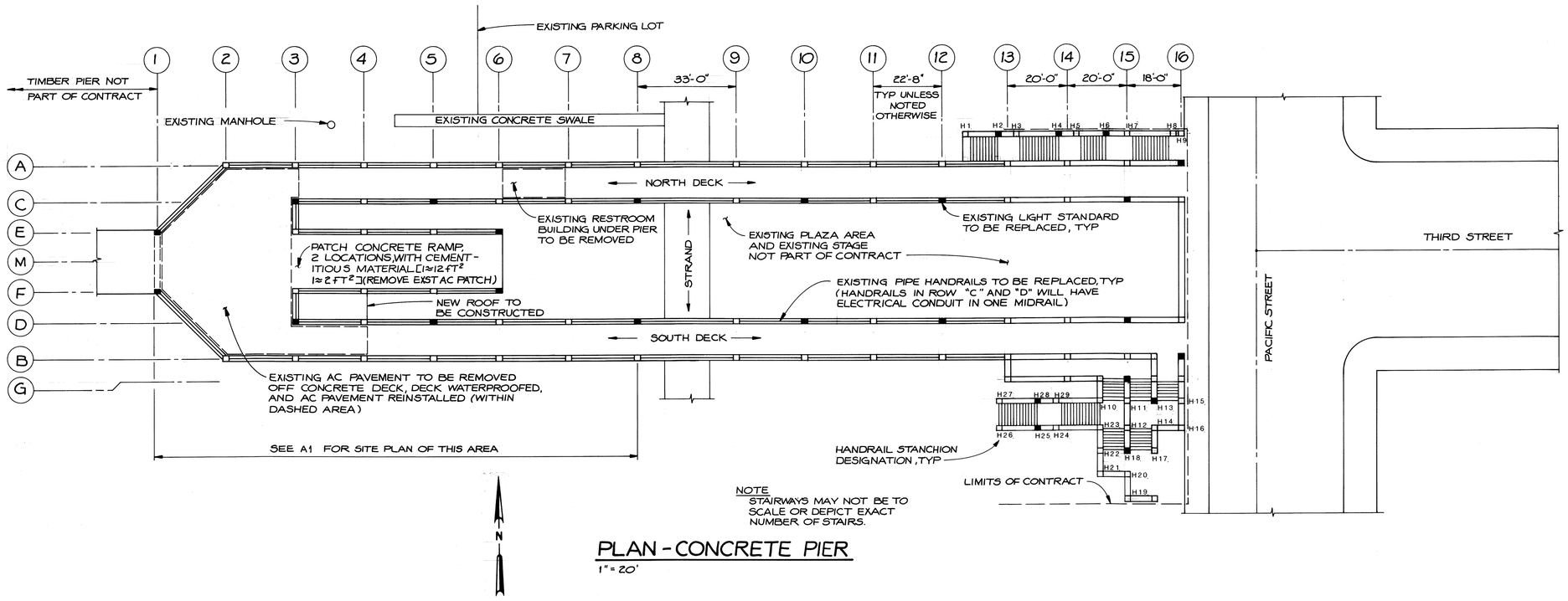
ENGINEER OF WORK
R.C.E. 20883

Checked by
Approval date

B-402

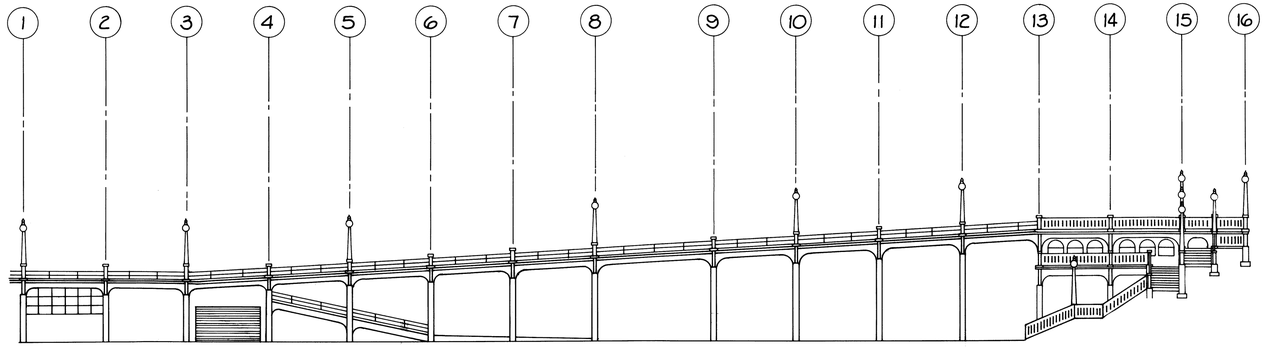
0000000154

89969



NOTE
STAIRWAYS MAY NOT BE TO SCALE OR DEPICT EXACT NUMBER OF STAIRS.

PLAN - CONCRETE PIER
1" = 20'



SOUTH ELEVATION
1" = 20'

NOTES:
1. Locations on pier requiring repair are noted in the field inspection report enclosed with the specification. They are referenced to the bent and row designation noted on these drawings. Areas found in possible need of repair that are not designated in the inspection report shall be brought to the attention of the Engineer.

2. All materials and construction shall be in conformance with these plans and specifications, the Standard Specifications for Public Works Construction (where not covered in these specifications), the City of Oceanside Department of Building and Safety, OSHA, and all other applicable codes and ordinances.

3. Dimensions and quantities shall be verified by the contractor.

APPROVED CHANGES

No.	Description	Approved By	Date

BENCH MARK
Description:
Location:
Record From:
Elev: Datum:

REGISTERED PROFESSIONAL ENGINEER
JAMES M. ROSS
20883
CIVIL
STATE OF CALIFORNIA

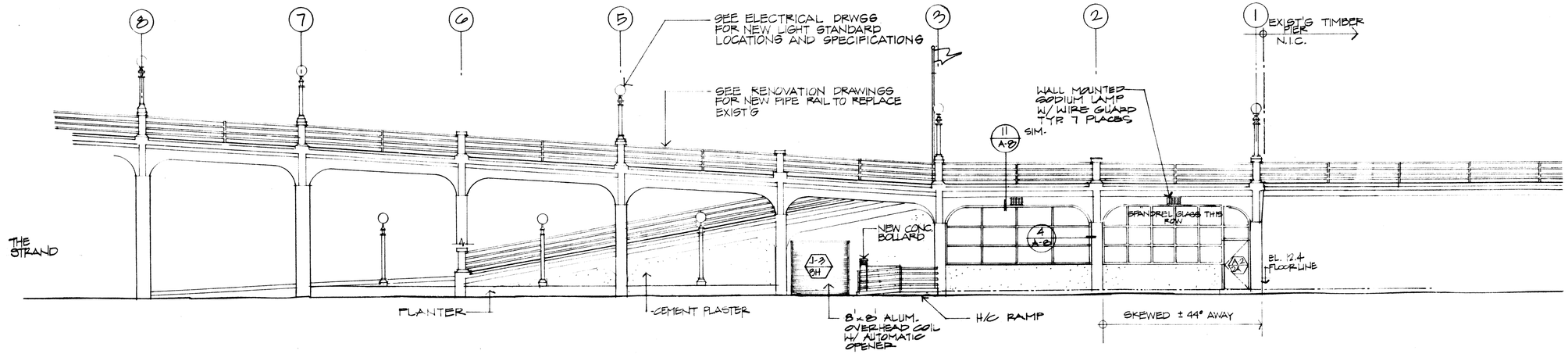
SHEET R-2 | CITY OF OCEANSIDE ENGINEERING DEPARTMENT | SHEETS

OCEANSIDE PIER RESTORATION
CONCRETE PORTION/LIFEGUARD HEADQUARTERS

PLAN AND ELEVATION

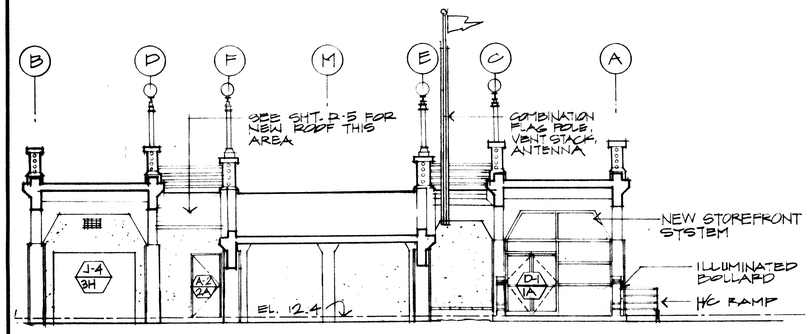
Approved Ronald A Beach 8-17-95 City Engineer

ENGINEER OF WORK R.C.E. 20883 | Checked by | Approval date | B-403



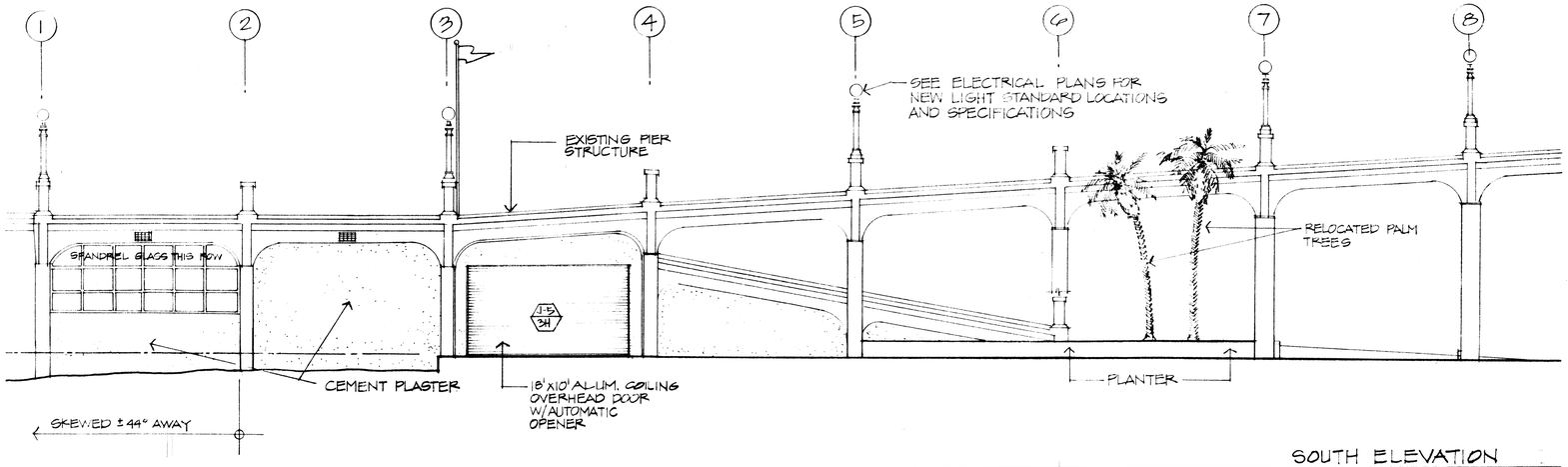
NORTH ELEVATION

SCALE 1/8" = 1'-0"



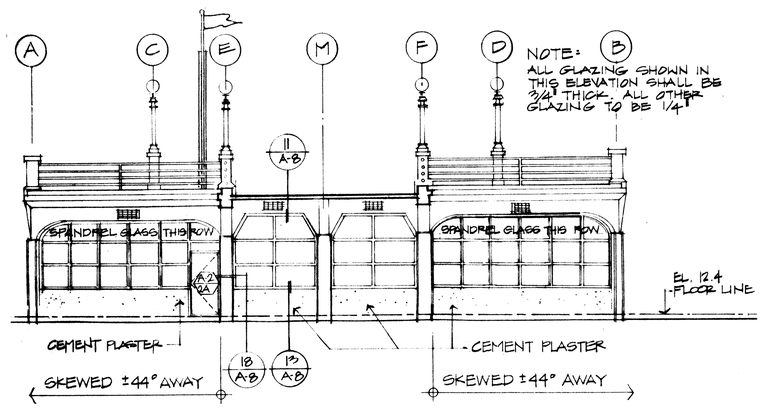
EAST ELEVATION

SCALE 1/8" = 1'-0"



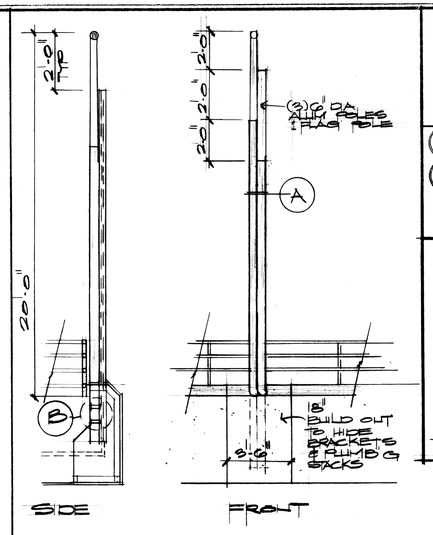
SOUTH ELEVATION

SCALE 1/8" = 1'-0"



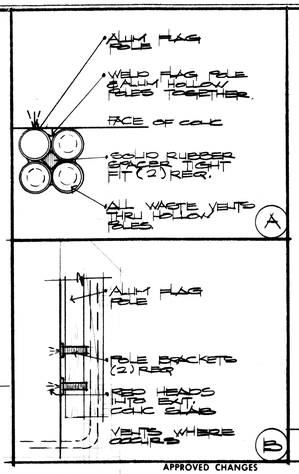
WEST ELEVATION

SCALE 1/8" = 1'-0"



FLAG POLE

1



HELVETICA MEDIUM

OCEANSIDE LIFEGUARD HEADQUARTERS

6" TYP.

DIE-CUT VINYL LETTERS TO BE INSIDE MOUNTED TO GLASS PANEL NEXT TO PUBLIC ENTRY DOORS SEE SPEC.

MOUNT ON DOOR TO RM. 111

MOUNT ON DOOR TO RM. 110

MOUNT ON DOOR TO RM. 102

THESE SIGNS TO BE ALUMINUM FRAME WITH ACRYLIC PLASTIC PLAQUE INSERTS SEE SPEC.

SIGNS

2

APPROVED CHANGES

No.	Description	Approved By	Date

BENCH MARK

Description: _____

Location: _____

Record From: _____

Elev: _____ Datum: _____

REGISTERED ARCHITECT — WILLIAM A. DANIELSON — No. C7396 — STATE OF CALIFORNIA

DANIELSON DESIGN GROUP

SHEET	CITY OF OCEANSIDE ENGINEERING DEPARTMENT	SHEETS
A-3		

OCEANSIDE PIER RESTORATION
CONCRETE PORTION/LIFEGUARD HEADQUARTERS

EXTERIOR ELEVATIONS

Approved Ronald A Beck 9-17-85 City Engineer

ENGINEER OF WORK	Checked by	
R.C.E.	Approval date	B-410