COASTAL DEVELOPMENT PERMIT APPLICATION NO. 21-04

STAFF REPORT

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FIGURES

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Figure 1. Project Location
Figure 2. Street Vacation - Portions of Southern Pacific Drive and Pennington Avenue
ATTACHMENT 1

POLB Water Quality Risk Due to Flooding Technical Memo
**Water Quality Risk Due to Flooding**

The portion of the Pier B On-Dock Rail Support Facility Project (Project) within the City of Los Angeles (COLA) dual permitting jurisdiction is located in an area of moderate flood hazard between the limits of the 100-year and 500-year flood level, meaning that there is an estimated 0.2 percent chance of a flood event in any given year (POLB, 2016).

**Construction**

Construction activities would result in minor reconfiguration of existing drainage basins that would redirect stormwater flows; the drainage system would safely and adequately convey flows to ensure that there would be no adverse effects to the area’s hydrology, floodplain and the overall elevation of the site would not be changed. There are no nearby levees or dams that would be subject to failure or expose people and/or structures associated with the Project.

There are industrial properties within the Project footprint (within the COLA) that are known to contain or handle hazardous materials, including the Valero Refinery and the BP Calciner Plant. Sites that contain or handle hazardous materials are responsible for compliance with all local, state, and federal rules and regulations, including the development and implementation of safety and emergency plans, Stormwater Pollution Prevention Plans (SWPPP), and spill prevention, control and countermeasure plans which are all designed to mitigate risks of hazardous materials releases. The Port is also required by law to ensure that a SWPPP for construction activities on construction sites over 1 acre is developed and implemented. The SWPPP shall include best management practices to control pollutants, sediment from erosion, stormwater, and non-stormwater runoff and other construction impacts. As standard practice, Project plans and specifications would be reviewed by the Los Angeles Fire Department for conformance to the Municipal Fire Codes. Construction personnel will also be required to participate in training of Port protocols in the event contaminated soils or groundwater are encountered during construction activities.

**Operations**

The portion of the Project within the dual coastal development permit jurisdiction would continue to operate in the same manner as under existing conditions. Project operations would not interfere with any existing contingency and emergency response plans; existing spill contingency and emergency response plans would be updated to incorporate operational changes.

The Project will include structures that will be located within the COLA dual permitting jurisdiction; these structures will be similar to existing structures in the Project area consisting of rail tracks, crash walls (State Route 103), track supports, pavement, track drainage, and lighting. These are not expected to induce or exacerbate flooding. The Project will be designed to not impede or redirect flood flows during site operations that would result in flooding. The storm drain design will safely and adequately convey flows when the rail yard is operating and ensure that there will be no adverse effect on the area hydrology or floodplain. In addition, operation of the Project would not change the existing topography, no slopes are proposed, existing soils are not erodible, and climatic conditions are relatively stable. The Project area would be ballasted, minimizing exposed soils. Therefore operation of the Project will not result in wind or water erosion that causes substantial runoff or deposition not contained or controlled.

Reference

ATTACHMENT 2

POLB Environmental Justice Memo
1. Introduction

The Port of Long Beach (Port) Pier B On-Dock Rail Support Facility Project (Project) will reconfigure, expand, and enhance the existing Pier B Rail Facility (primarily located within Long Beach Harbor District) to allow longer trains to be loaded and unloaded at the on-dock rail facilities at the Port’s marine terminals to streamline operations, remove bottlenecks in the rail system, and reduce the need for local truck trips. By using on-dock rail, for every vessel call, approximately 3,750 truck trips can be replaced by loading cargo directly onto trains at the marine terminals. Each 10,000-foot, double-stacked train loaded on-dock at the Port would replace up to 750 truck trips on local freeways, reducing the number of trucks on regional roadways and freeways. These trucks would otherwise transport cargo containers to the near-dock or off-dock rail yards, where the containers would need to be individually handled to be unloaded from each truck using additional cargo-handling equipment and then loaded onto trains at those yards by cargo-handling equipment prior to departure to their destination.

The Project fulfills the Port’s mandates under the State of Tidelands Trust to promote and develop commerce, navigation, and fisheries, and other uses of statewide interest and benefit, including industrial and transportation uses and Chapter 8 of the California Coastal Act (CCA), which recognizes California Ports, including the Port of Long Beach as primary economic and coastal resources that are essential elements of the national maritime industry (Section 30701(a)). Consistent with Section 30708(a) of the CCA, the Project will minimize substantial adverse environmental impacts by facilitating the transport of a larger proportion of containerized cargo directly to and from the Port by rail instead of drayage trucks. This supports the Port’s Clean Air Action Plan (CAAP) Health Risk Reduction Standards, Emissions Reduction Standards, and Operational Efficiency Improvement Initiatives, and the State’s Sustainable Freight Strategy.

The Project would also utilize existing lands and maximize the use of existing and proposed port rail infrastructure, thereby promoting maritime commerce. This is consistent with Section 30708(c) of the CCA. In accordance with Section 30708(e) of the CCA, the Project will encourage rail service to Port areas and multi-use of facilities by assisting the marine terminals in optimizing their operations by allowing more cargo to be transported by rail companies such as Union Pacific Railroad, Burlington Northern Santa Fe, and Pacific Harbor Line, all of which operate in the San Pedro Bay Ports and the existing Pier B rail yard.

2. Project Overview

The Project site is primarily located in the Long Beach Harbor District, however a small portion (approximately 25 percent) of the Project is sited in a portion of the Wilmington-Harbor City Community Plan Area of the City of Los Angeles (COLA). The proposed Project area includes rail tracks that extend
west beyond the Terminal Island Freeway (State Route [SR] 103) to just west of the Dominguez Channel, where they connect with the Alameda Corridor. The portion of the project within the COLA is a dual coastal development permit jurisdiction with the California Coastal Commission (CCC). Figure 1 (below) shows the portion of the Project within the dual coastal development permit jurisdiction. The following components of the Project will occur in the COLA portion, an existing, dedicated rail right-of-way dominated by heavy and light industrial and port-related industries:

- Railroad tracks
- Addition of railroad track to the Dominguez Channel Rail Bridge
- Crash walls (State Route 103)
- Track supports (to protected existing bridge footings)
- Pavement (access roads)
- Electrical (lighting for rail corridor)
- Compressed air (for train brakes)
- Utility relocations
- Track drainage
- Fire hydrants (locomotive facility)
- Minor street improvements

Figure 1. Pier B On-Dock Rail Support Facility Project – Overview of Design Elements within the City of Los Angeles Jurisdiction

3. Harbor Development Permit and Environmental Review

Pursuant to the requirements established by the CCA, the CCA granted coastal permitting authority for development projects (Coastal Development Permit [CDP]) within the Long Beach Harbor District to the Long Beach Board of Harbor Commissioners (BHC). A Harbor Development Permit (HDP), a consolidated CDP and Building Permit per Section 1215 of the Long Beach City Charter, was approved for the Project by the BHC in January 2018 for the majority of the Project located within the Long Beach Harbor District. As discussed previously, the Project was determined to be consistent with both the Port of Long Beach Certified Port Master Plan and the CCA. Figure 2 (below) shows the portion of the Project area in
relation to the City of Los Angeles and City of Long Beach boundaries. No public comment was received from the CCC or COLA during the public hearing.

Figure 2. Pier B On-Dock Rail Support Facility Project – Overall Project Area in Relation to the City of Los Angeles and City of Long Beach Boundaries

**CEQA**

Prior to issuance of the HDP, as the Lead Agency under the California Environmental Quality Act (CEQA), the Port ensured opportunities for coordination with the public, organizations, and agencies throughout the preparation of the Environmental Impact Report (EIR), which included the release of the Notice of Preparation (NOP) and Initial Study in August 2009. The NOP described the proposed Project and its potential environmental effects, soliciting comments from the public and responsible agencies such as the COLA and CCC on the scope of the environmental analysis that should be included in the EIR. The COLA and the CCC were both identified as responsible agencies because of their authority to issue the necessary permits for the minimal portion of the Project within the COLA jurisdiction. Responsible agencies comply with CEQA by considering the EIR prepared by the lead agency and by reaching their own conclusions on whether and how to approve the Project. No comments were received in response to the NOP from either the COLA or California Coastal Commission.

A Notice of Completion and Availability of the Draft EIR were issued by the Port in December 2016, providing the public, as well as governmental and regulatory agencies, an opportunity to review the Draft EIR, the potential environmental impacts and proposed mitigation measures, and provide input on the environmental analyses. Three public meetings to solicit comments on the Project were held on January 11, 2017, January 18, 2017, and February 15, 2017. The public meetings were scheduled in the evenings at locations accessible by public transit and with adequate parking, including the Port Interim Administration Offices, Silverado Park, and during the mid-day at Tepechi Birreria Restaurant located adjacent to the North Long Beach Harbor District in West Long Beach. During each of the public meetings, members of the public were provided an opportunity to provide oral comments. Translators were available to provide American Sign Language and Spanish translation services. In addition, the Port
met with environmental groups, Long Beach community and neighborhood groups, as well as community groups in Wilmington prior to the certification of the Final EIR, including the Wilmington Neighborhood Council Executive Board on February 6, 2017, Wilmington Chamber of Commerce on February 9, 2017, and the Wilmington Neighborhood Council Board and Stakeholders at Banning High School on February 22, 2017. All comments received on the Draft were considered and the Port’s responses are documented in Chapter 11 of the Final EIR for the Project. After the 90-day review and comment period, over 300 comments were received from 48 governmental and regulatory agencies, organizations, and members of the public. The Port considered and provided responses to each comment received on the Draft EIR, which were incorporated into the Final EIR certified by the Long Beach BHC in January 2018.

The Final EIR concluded that although most potentially significant impacts resulting from the Project are rendered less than significant through mitigation measures, construction and operation of the Project would produce air quality and greenhouse emissions (GHG) impacts that are significant and unavoidable. Pursuant to CEQA, there are specific overriding economic, legal, technological, and other benefits of the Project that outweigh the significant impacts and provide important reasons for implementation of the Project. The Project will:

- Fulfill the Port’s mandates under the State Tidelands Trust to promote and develop commerce and navigation, and other uses of statewide interests and benefit, including industrial and transportation uses;
- Meet the California Coastal Act’s provision that the Port should give the highest priority to existing land space within harbors for port purposes by maximizing the use of existing and proposed rail infrastructure in the port complex;
- Support the San Pedro Bay Ports CAAP initiative to maximize the use of on-dock rail to meet the Port’s emission reduction goals. The Project is consistent with the CAAP and includes all applicable CAAP measures;
- Support the State’s Sustainable Freight Action Plan to improve freight efficiency and make California’s freight system more competitive;
- Implement the Regional Transportation Plan to transport cargo via rail versus truck, thereby reducing vehicle miles traveled.

After the filing of the Notice of Determination and exhaustion of the statute of limitations for any legal challenges, the CEQA process was completed in April 2018. The Draft and Final EIR documents for the Project are available on the Port’s website at [https://www.polb.com/ceqa](https://www.polb.com/ceqa).

### 4. Environmental Justice and Disadvantaged Communities

While an analysis of Environmental Justice is not required under CEQA, the certified EIR includes an evaluation of the potential for the Project to cause environmental effects on low income and minority populations to satisfy the COLA environmental justice policy adopted in the Framework and Transportation Element of its General Plan. Disadvantaged communities adjacent to the Port experience higher rates of pollution and generally consist of a higher proportion of minority and low-income populations. Based on an evaluation of 2010 Census data, the EIR for the Project identified that within a one-mile radius of the project, the proportion of the minority population was 83.5% while 28.1% fell within the low-income population. In the COLA, with a population of 3,792,621, 45% was minority and
21.2% low-income. The City of Long Beach’s (COLB) population of 462,257 was comprised of 38.2% minority and 20.2% low-income. The County of Los Angeles had an overall population of 9,818,605, of which 45.3% was minority and 17.1% low-income. For the purposes of the evaluation, the percentage of minority and low-income populations in the COLA and COLB were considered to be meaningfully greater than that of the general population in Los Angeles County.

While the area surrounding the Project site is primarily industrial, there are residences in the Wilmington Community and in the COLB that include predominately minority and low-income populations. Under Assembly Bill 617, the California Air Resources Board established the Community Air Protection Program, identifying the Communities of Wilmington, Carson, and West Long Beach collectively as a disadvantaged community. In September 2019, the South Coast Air Quality Management District (SCAQMD) adopted the Community Emissions Reduction Plan (CERP) for the Wilmington, Carson, and West Long Beach Communities, developed in partnership with residents and community stakeholders which outlines “actions” to reduce air pollution emissions or exposures. The actions identified in the CERP include the ongoing efforts to reduce emissions from goods movement activities at the ports of Long Beach and Los Angeles, including compliance with state regulations and implementation of measures and strategies identified in the 2017 Update to the San Pedro Bay Ports Clean Air Action Plan (CAAP). This Project is vital to support the 2017 CAAP Update planning and investment initiative to transform the Port’s infrastructure to support on-dock rail and supply chain efficiencies. In addition, the Port continues to demonstrate near-zero and zero-emission technologies for port-related mobile sources, including locomotives. Port of Long Beach staff participated in SCAQMD’s development of the CERP and expressed support for the strategies outlined in the CERP. The Port continues to monitor the efforts of the CERP and implement its own initiatives and measures under the CAAP, such as on-dock rail infrastructure, to reduce air emissions from port-related sources.

5. Environmental Impacts to Low-Income and Minority Populations and Mitigation Measures

As previously discussed, the Project would provide a facility within the Port dedicated to supporting more on-dock rail operations and improving the overall efficiency of goods movement within the Port and on the regional transportation network, the CAAP, the San Pedro Bay Emission Reductions Standards, and the State’s Sustainable Freight Action Plan.

The EIR for the Project concluded that it would be reasonable to assume that residual significant and unavoidable impacts to air quality associated with construction and operation of the Project could disproportionately affect low-income and minority populations near the Project site. It is important to note that the EIR conservatively evaluated the effect of additional trains associated with the operation of the Project, and did not take credit for any air quality benefits from the replacement of truck trips that would result from the project. As such, the impacts associated with the Project may be lower than estimated in the EIR.

The EIR identified the maximum off-site locations that would be impacted by ambient air pollutant concentrations associated with construction and operation of the Project. The maximum off-site

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1 Assembly Bill 617, 2017, C. Garcia, Chapter 136, Statutes of 2017. Requires the California Air Resources Board and Air Districts to develop and implement additional emissions reporting, monitoring, reduction plans, and measures to reduce air pollution exposure in disadvantaged communities.
locations, for the Project, have been identified to be directly on the Project boundary.

**Construction**

Construction activities have the potential to generate temporary pollutant emissions in the COLA. Exceedance of the 1-hour NO₂ standard (federal) and Annual NO₂ standard would occur at a maximum-impacted location in the COLA during Phases 1 and 2 of construction activities and exceedance of the Annual NO₂ standard would occur during phase 3 of construction activities. Mitigation Measures AQ-1 through AQ-5 (see Table 1) would reduce the ambient air quality impacts during construction; however impacts would remain significant and unavoidable. It is important to note that while the analysis indicated that NO₂ emissions would exceed the ambient air quality standards, this is primarily due to the already high background concentrations of NO₂ in the air basin, leaving very little room for the estimated incremental NO₂ emissions associated with the Project to stay under the significance thresholds.

**Operations**

During Project operations, exceedances of the federal 1-hour NO₂ standards would occur in years 2020, 2025, and 2035 at maximum impacted locations in the COLA. The Annual NO₂ concentration would exceed the significance thresholds in year 2020 and 2025, but not 2035. This is due primarily to the transition to cleaner locomotives. It is also important to note, the modeling to determine the maximum NO₂ impacts is very conservative because it assumes that the peak hour background concentration of NO₂ would hold steady at the highest level during the entire year, and rail yard activity would occur at peak levels every hour of the year—which would not occur during actual operations.

There are no feasible mitigation measures identified for the Project’s operational impacts. However, to reduce cumulative air quality impacts associated with operation of the Project, among the mitigation measures that are required to be implemented pursuant to CEQA includes Mitigation Measure AQ-6, which aims to reduce cumulative air quality impacts associated with Project operation through the contribution of $149,757 to the Port’s Community Grants Program. In addition, as a Special Condition for issuance of the Harbor Development Permit for the Project, the Port is required to conduct a review of new air quality technology advancements for operational feasibility, technical feasibility, cost effectiveness, and financial feasibility for application in the Pier B On-Dock Rail Support Facility.

Unlike criteria and toxic air contaminants, the effects of GHG emissions are not specific to the area surrounding the Project; rather, it is a cumulative effect on a global scale. Therefore, GHG emissions would not adversely affect the surrounding populations to a greater degree than elsewhere. However, because GHG emissions associated with the Project are considered to be cumulatively significant and unavoidable, Mitigation Measure GCC-8 requires the Port to provide $1.4 million to the Community Grants Program to partially address the cumulative GHG emissions impacts of the Project. The Port’s Community Grants Program was established to provide funding for projects that will help to reduce air quality impacts to vulnerable groups in the Port area and reduce GHG Emission that contribute to global climate change.

Table 1 provides a summary of all adopted air quality and global climate change (GCC) mitigation measures and special conditions. As the lead agency, the Port is responsible for administering and implementing the mitigation measures.
Table 1. Pier B On-Dock Rail Support Facility Project
Air Quality and Global Climate Change Mitigation Measures and Special Conditions

<table>
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<tr>
<th>AIR QUALITY MITIGATION MEASURES</th>
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<tr>
<td><strong>Mitigation Measure AQ-1: On-Road Construction Trucks.</strong> All on-road heavy-duty trucks with a fifth-wheel tractor/trailer and a gross vehicle weight rating (GVWR) of 19,500 pounds or more transporting materials to and from the construction site shall meet United States Environmental Protection Agency (EPA) 2010 on-road heavy-duty diesel engine emission standards.</td>
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<tr>
<td><strong>Mitigation Measure AQ-2: Tier 4 Construction Equipment.</strong> All self-propelled, diesel-fueled off-road construction equipment 25 horsepower (hp) or greater shall meet EPA/California Air Resources Board(CARB) Tier 4 off-road engine emission standards.</td>
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<td><strong>Mitigation Measure AQ-3: Off-Road Construction Equipment.</strong> Off-road diesel-powered construction equipment shall comply with the following:</td>
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| Maintain all construction equipment according to manufacturer’s specifications.  
| Construction equipment shall not idle for more than 5 minutes when not in use.  
| High-pressure fuel injectors shall be installed on construction equipment vehicles.  |
| The benefits to be achieved by the above-listed components of this measure were not quantified in the analysis due to the wide range of variables involved. This measure is applied, however, to further reduce combustion emissions. |
| **Mitigation Measure AQ-4: Increased Watering Frequency for Fugitive Dust Control.** Construction site watering, required by SCAQMD Rule 403, shall be increased such that the watering interval is no greater than 2.1 hours. This measure would increase the fugitive dust emissions control from 61 to 74 percent. |
| **Mitigation Measure AQ-5: Additional Fugitive Dust Control.** Contractors shall: apply approved nontoxic chemical soil stabilizers according to manufacturers’ specifications to all inactive construction areas or replace ground cover in disturbed area; Provide temporary wind fencing around sites being graded or cleared; Cover truck loads that haul dirt, sand, or gravel or maintain at least 2 feet of freeboard in accordance with Section 23114 of the California Vehicle Code; Install wheel washers where vehicles enter and exit unpaved roads onto paved roads, or wash off tires of vehicles and any equipment leaving the construction site; Suspend all soil disturbance activities when winds exceed 25 miles per hour (mph) or when visible dust plumes emanate from the site and stabilize all disturbed areas. |
| **Mitigation Measure AQ-6: Cumulative Air Quality Impact Reduction Program.** To reduce air quality impacts associated with operation, the Port will contribute to the Community Grants Program (CGP). For the proposed Project, the contribution to the CGP would be $149,757 total. |

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<tr>
<th>AIR QUALITY SPECIAL CONDITION</th>
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<tr>
<td><strong>Periodic Technology Review:</strong> Every 5 years following the Project approval date, the Port shall conduct a review of new air quality technological advancements. These technologies would be evaluated on operational feasibility, technical feasibility, cost effectiveness, and financial feasibility for application in the Pier B Rail Yard. If a technology is determined to be feasible in terms of financial, technical, and operational feasibility, the Port shall implement such technology.</td>
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<tr>
<th>GLOBAL CLIMATE CHANGE/GREENHOUSE GAS EMISSIONS</th>
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<td><strong>Mitigation Measure GCC-1: LEED.</strong> New buildings constructed as part of the proposed Project shall meet COLB Green Building Policy criteria, and Leadership in Energy and Environmental Design (LEED) certification shall be sought. COLB exempts buildings of less than 7,500 square feet of occupied space from its Green Building Policy.</td>
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<tr>
<td><strong>Mitigation Measure GCC-2: Recycling of Construction Materials.</strong> Construction debris must be recycled, reused or otherwise diverted from landfills to the maximum extent possible. Recyclable construction waste generated by the Project shall be taken to an accredited recycling center.</td>
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<tr>
<td><strong>Mitigation Measure GCC-3: Recycling and Sustainable Business Practices.</strong> During operation, the Port shall follow recycling objectives and measures established by the Port’s Administrative Directive (Sustainable Business Practices, including energy conservation practices, purchasing of “Green” products, energy-efficient lighting, low-volatile organic compound (VOC) paint and finishes, and use of recycled or remanufactured carpeting and office furnishings, minimized use of paper and plastic, reuse of materials and equipment, and proper disposal of alkaline batteries.</td>
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<td><strong>Mitigation Measure GCC-4: Xeriscaping.</strong> Xeriscape landscaping incorporating the use of water conservation features including, but not limited to, drought-tolerant plants; hardscape; permeable material such as concrete, asphalt, and pavers; recycled material such as concrete, gravel, granite, and shredded redwood; and drip irrigation systems and timers.</td>
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<tr>
<td><strong>Mitigation Measure GCC-5: Tree Planting.</strong> The Port shall plant shade trees around the main office and maintenance buildings in accordance with species identified in the Green Port of Long Beach Sustainable Landscape Palette and POLB Sustainable Development Guidelines.</td>
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</tbody>
</table>
**Mitigation Measure GCC-6: Tree Planting – Transportation Corridors.** The Port shall plant new shade trees on Port-controlled lands adjacent to the roads that lead into the facility, to the extent practicable, consistent with safety and other land use considerations.

**Mitigation Measure GCC-7: Employee Carpooling.** The construction contractor and the Port shall encourage construction and facility employees to carpool or to use public transportation. These employers shall provide incentives to promote the measure, such as preferential parking for carpoolers or vanpool subsidies, and they shall provide information to employees regarding the benefits of alternative transportation methods.

**Mitigation Measure GCC-8: Community Grants Program (CGP).** The Port will implement and fund the CGP to partially address the cumulative GHG impacts of the proposed Project. The Port shall provide $1.4 million, as determined by the POLB CGP funding level methodology.

**Mitigation Measure GCC-9: Indirect GHG Emission Avoidance and Mitigation.** The Port shall minimize indirect GHG emissions through measures that reduce or avoid electricity consumption at the facility. Such measures may include, but are not limited to, the use of low-energy demand lighting (e.g., fluorescent or light-emitting diode [LED]), and use of energy-efficient floodlights. To identify future opportunities to reduce indirect GHG emissions, the Port shall conduct a third-party energy audit every 5 years and install innovative power-saving technologies where feasible, such as power factor correction systems and lighting power regulators. Such systems help to maximize usable electric current and eliminate wasted electricity, thereby lowering overall electricity use.

6. **Port of Long Beach Community Efforts**

The Port, guided by its landmark Green Port Policy, proactively plans for the future by modernizing facilities and infrastructure and creating operational efficiencies where possible; ensuring that the Green Port Policy is reflected in land use planning and policies; prioritizing safety and security; providing and enhancing access to information, educational opportunities, recreational amenities, and cultural experiences; and advancing environmental justice for communities surrounding the Port. As the nation’s second busiest seaport and a vital economic engine for the City of Long Beach and the region, the Port recognizes that economic vitality comes at a cost to the local community, which bears the brunt of the environmental and public health impacts of Port development and operation.

**Public Engagement**

A fundamental element of the Port’s Green Port Policy is community engagement to interact and educate the community on the Port’s development and environmental programs. The Port places a strong emphasis on meaningful, comprehensive engagement with local communities surrounding the Port and other stakeholders to educate and inform through a wide-ranging communications outreach program that offers sponsorship, partnership and educational opportunities to enhance communities’ understanding of Port operations and initiatives, environmental programs, development projects, and to encourage participation in the decision-making process. The Port engages regularly and directly with local community groups, environmental justice leaders, and the public through recurring meetings, social media, e-mail blasts, and website notifications.

For the Project, the Port holds quarterly Pier B On-Dock Rail Support Facility Project Stakeholder Outreach meetings to provide the latest updates on the Project’s status. The Stakeholder Outreach meetings are typically held at locations adjacent to the Project footprint. The Port has also provided presentations to the Coastal, Central, Northwest San Pedro Neighborhood Councils, in which representatives from the Wilmington Neighborhood Council participated and provided remarks at the Coastal San Pedro Neighborhood Council meeting on April 19, 2021. In addition, on May 10, 2021, the Port’s Deputy Chief Harbor Engineer provided a presentation on the Project at the Wilmington Neighborhood Council’s Special Joint Committee Meeting of the Planning & Land Use and Beautification Committees and provided remarks to at the Neighborhood Council’s Governing Board meeting on May 10, 2021. Also on May 10, 2021, staff from the Port’s Environmental Planning Division held a call with a
representative of the Wilmington Neighborhood Council to discuss the potential environmental impacts associated with the Project and the Port’s Community Grants Program.

**Community Grants Program**

Through the Port’s award-winning Community Grants Program, the Port coordinates with local residents and communities to foster communication and collaboration to address areas near the Port that are most impacted by Port operations. Since 2009, the Port has set aside nearly $65 million for grants to improve the health of children, seniors and other vulnerable populations, reduce GHG emissions, and enhance the environment. Projects funded under the program have included development of public parks, water quality improvements, light-emitting diode (LED) lighting, healthcare programs, facility-based energy efficiency projects, trees and landscaping, solar panels, electric vehicles, air filtration, and noise abatement measures. As previously discussed, the Project will contribute a total of nearly $1.5 Million to the Community Grants Program to address the cumulative impacts to air quality and global climate change. The Port has established criteria for eligible projects and programs and the types of organizations that can apply, in accordance with the public trust doctrine and guidance from the California State Lands Commission. The Community Grants Program has identified areas that have experienced the highest impact from Port-related operations. In addition to the City of Long Beach, the communities of Wilmington, Carson, Compton, and Paramount are encompassed within the geographic area identified by the Community Grants Program to apply for grant funding. A Community Grants Advisory Committee appointed by the Mayor of Long Beach assists in selecting projects for funding. Before any funding is awarded, the Port conducts a thorough review of all applications prior to presenting them to the Long Beach BHC for approval.

The Port also provides community sponsorships that increase the Port’s visibility and engagement while expanding our reach into the local community. The hundreds of local nonprofit recipient groups each year represent a cross-section of the region. Sponsorships support causes such as the arts, environment, social justice and historic preservation. Through sponsorships, the Port partners with groups in events and activities that provide the Port with opportunities to educate and inform key audiences about Port projects and programs.

The Port’s educational outreach creates awareness among students of the Port’s role in the local, regional, and national economies and its role in the international goods movement industry. The Port fosters opportunities for students to learn about careers in trade, and provides support to students to pursue the education and training needed for those careers through partnerships with the Long Beach Unified School District and Long Beach City College. The Port’s Academy of Global Logistics at Cabrillo High School in West Long Beach is a four-year program that combines an academic curriculum with industry-relevant training and information to support academic and career development. The Port has also partnered with Long Beach City College to provide training opportunities to individuals seeking to develop or enhance their knowledge and skills in the supply chain/logistics sector.

**Access to Coastal Resources**

The Project is located in an area primarily dominated by heavy and light industrial and port-related industry. However, the Port seeks to balance public access and recreation opportunities throughout the Long Beach Harbor District. In May 2021, the Pier J South Waterfront Bicycle Path project was completed and opened to the public, extending an existing bicycle path roughly 2 miles into the Queen
Mary complex and into areas of the Port’s waterfront that are suitable for public access. This bicycle path provides visitors a panoramic view of the downtown Long Beach skyline, marinas, beaches, and Long Beach Harbor. Other bicycle and pedestrian facilities in the Port will include the Mark Bixby Memorial Bicycle and Pedestrian Path on the new Long Beach International Gateway Bridge (Gerald Desmond Bridge Replacement) and the Ocean Boulevard Bicycle Gap Closure Project. Options for connections to the California Coastal Trail in the North Long Beach Harbor Area are also being evaluated by the Port.

7. Conclusion
The Pier B On-Dock Rail Support Facility Project is a critical junction in the Port’s goods movement system and will promote the transition to a more efficient, more economically competitive, and less polluting freight transport system through the transportation of containerized cargo directly to and from the Port via rail instead of by drayage trucks. The Project supports mandates under the State’s Tidelands Trust that requires the Port to promote and develop commerce, navigation, and fisheries, and other uses of statewide interest and benefit, including industrial and transportation uses. The California Coastal Act also recognizes California Ports, including the Port of Long Beach, as primary economic and coastal resources that are essential elements of the national maritime industry. The Coastal Act obligates the Port to modernize and construct necessary facilities to encourage rail service to Port areas and multi-company use of facilities.

The environmental analyses for the Project are very conservative. While the EIR identifies that it is reasonable to assume that the Project would have residual significant and unavoidable impacts to air quality disproportionately affecting low-income and minority populations near the Project site, the Project may actually produce air emissions and health impacts that are less than the values presented in the EIR with the replacement of drayage truck trips with rail trips from the marine terminals served by the Pier B On-Dock Rail Support Facility. The Port is committed to implementing all feasible mitigation measures to ensure that environmental impacts associated with the Pier B On-Dock Rail Support Facility Project are reduced, while also ensuring that the Green Port Policy is reflected in the Port’s efforts by providing meaningful engagement with the public; access to information, educational opportunities, and access to coastal resources, while advancing environmental justice for communities surrounding the Port.
ATTACHMENT 3
WNC Letter, dated May 25, 2021
May 25, 2021

Mayor of Long Beach, Robert Garcia
Mayor of Los Angeles, Eric Garcetti
Port of Long Beach, Harbor Commissioners
Port of Long Beach, Environmental Planning Department
Los Angeles City, Planning Department
Los Angeles City Council
Congresswoman, Nanette Barragan
AQMD

Subject: CF 19-0739, Coastal Permit Case# DIR-2020-7285-CDP
Oppose the Long Beach Port, Pier B On-Dock Rail Support Facility Project

Dear Honorable Leaders,

The Wilmington Neighborhood Council Governing Board held a public Brown Act meeting to discuss the Port of Long Beach, Pier B On-Dock Rail Support Facility Project. After a presentation was given by the Port of Long Beach, the WNC Planning & Land Use committee reviewed the documentation from the Los Angeles City Planning Department. Upon further review we recommend the following:

We oppose this current and ongoing Long Beach Pier B On-Dock Rail Support Facility Project that will have severe impacts on our stakeholders and take land away from our community. Although presentations were given to the Neighborhood Councils in San Pedro with requests for letters of support for this project, this rail project does not go through the community of San Pedro at all. The project only goes through the community of Wilmington which is completely separate from San Pedro. The location of this expansion flows into Wilmington from the Long Beach borders. San Pedro will not hear or feel the impacts of this project. They are approximately 6.8 miles away on the other side of the bay.

Here is the summary of our cost benefit analysis:

**Costs to the Wilmington Community:**
The Final EIR concluded that the project would pose “significant and unavoidable air quality and health risks and greenhouse gas emission would remain higher than the SCAQMD threshold”. (File #:HD-18-034, Version: 1, page 5 of 6)
The 24-hour sound emissions and ground vibrations from this project may “average out” to be within the Federal standards, but this does not reflect the reality of the negative impacts of sleep deprivation on a community that is already living in one of the most challenging environments in our city.

There will be increased fire and explosion risks due to the refueling of locomotives. Wilmington will be losing more land that could be used for local businesses and jobs.

Benefits to the Wilmington Community:
The project will deposit $1.45 million over seven years to the special fund for mitigation. This is only 0.16% of the project’s $900M budget and can only be accessed through a competitive grant process. In comparison, other commercial development projects in the City of LA must contribute 1% toward public art.
The general “more jobs” benefit was mentioned during the presentation. However, based on experience most of those working in port related jobs choose to live and shop outside of Wilmington.

Conclusion
The costs of this project outweigh the benefits for our community of Wilmington. The idea that a port expansion project of this magnitude would not proactively identify and mitigate its negative impacts on our community is appalling.

Recommendations
Port expansion projects should include a community impact and mitigation study conducted by an independent third-party expert. The Ports have not demonstrated the ability to fully understand and mitigate the negative impacts to our Wilmington community. A third-party expert is necessary to identify what mitigation is necessary and to propose the best use of funds to offset the negative impacts of the port expansion. Determining the solutions for mitigation should not be put on the community and the funding should not be doled out through competitive grants. The ports should proactively think like a community member that raises their families here and needs to bear the endless noise, traffic, pollution, blight, and then crime and drug use that festers out of these conditions.

Here are some examples of the types of mitigation measures that a third-party expert may determine to be appropriate:

- Double pain windows
- HVAC systems with high quality filtration systems
- Renewable energy systems to power the HVAC systems
- Code enforcement for port related traffic and storage
- Creating more buffer zones between residential and all port related industrial activities
- Recurring periodic cleaning of homes and vehicles of port related industrial dust

Industrial commerce, in the name of economic development for the city, State and Nation, directly leads to hazards in our community such as, crumbling roads and streets, port truck traffic, noise, air, trains, water and land pollution. This community is a coastal community with contaminated ocean waters, no safe beach access or community coastal access. No views due to the increased Port growth, unsightly
container storage yard, and port cranes. Our underprivileged community is in great need of basic resources. We are now blighted by the Port’s impacts.

These issues intersect with our daily lives and causes unfair burdens. It all makes for a more dangerous and unsafe community. The health and well-being of those who live here are in great danger of contracting asthma, bronchitis, even cancer. Residents who live in Wilmington have cancer among other health issues due to the environmental hazardous directly related to port businesses. Wilmington is one of the Nation’s most polluted communities and only one of three communities in Los Angeles who fall under the Clean up Green up ordinance. All due to the port and related businesses.

The goal to eliminate the number of trucks on the road and increase containers to be loaded onto trains can never be totally achieved. (30%) of cargo is loaded onto trains. The cargo that is loaded onto these rail cars is cargo going to the center of our Nation. It will always be the smaller percentage of cargo received from vessels docked in our ports.

“Local” cargo discharged from port vessels are trucked (70% of cargo) and it is the largest percentage in volume that can never be eliminated. This trucked cargo goes to local cities and they will always need supplies which are labeled “Local Loads” going to nearby cities and even neighboring states such as Arizona, Nevada and Utah. This trucked cargo will still go through our communities, and as the Port grows, so does the trucked cargo.

Community impacts
- The Environmental Impact Report states that pollution levels will increase.
- The Environmental Impact Report states that the impacts are great and unmeasurable.
- The Environmental Impact Report states that the impacts are significant and unavoidable.
- The city of Los Angeles directly mitigates impacts from LAX with local stakeholders. This is a transportation mitigation and should be handled in a similar way. Direct.
- The Ports of LA/ LB and the cities of Los Angeles & Long Beach continue to reap the monetary benefits without proper community mitigation
- Wilmington sits on the third largest oil field in the Nation, subject to methane gas and oil wells.
- The project report states that 30 locomotives will be fueled in this area which poses a safety hazard. With several large refineries, the impacts are deadly and unwelcomed by stakeholders who travel this area.
- Our low-income community of color is overburdened.

The Port of Long Beach has been enjoying the benefits of the Port expansion with record breaking numbers each year. The May 2021 report states that the Port of Long Beach had the strongest April in history with a 43 percent increase and for the 10th consecutive month the Port has broken monthly cargo movement records. With these figures, the Port’s economy is booming.

Our Nation’s reliance on maritime transportation and international trade remains unchanged as there is the essential need for cargo to move through our ports. The maritime industry has kept our supply chain functioning and our economy strong but it is time to finally take responsibility, time to address the EIR effects imposed on our community. It is time to address the health risk associated with living next to a Port industrial complex. Further it is time to improve addressing all these issues.
Action

Due to the harmful impacts the project has on our community:

1) We urge LA City Council to **deny the Coastal Development Permit**
2) We ask LA City Council to **assert jurisdiction** over this matter and address these serious concerns
3) Please ask the port to hire a third-party expert who can identify what mitigation is necessary and to propose the best use of funds to offset the negative impacts of the port expansion directly. Determining the solutions for mitigation should **not** be put on the community of Wilmington.
4) We ask for a moratorium to be placed for the next 10-20 years to collect data on the extent of the impacts this project is having on our community.

As the duly elected body, by way of the city charter, the Wilmington Neighborhood Council is grateful for the opportunity to advocate. We are recognized as volunteer elected officials as we serve both the City of Los Angeles and our community through the Neighborhood Council System.

Respectfully Submitted,

Gina Martinez, Chair of the Wilmington NC
On Behalf of the Wilmington Neighborhood Council
ATTACHMENT 4
POLB Sea Level Rise Technical Memo
1 – Introduction

To maintain its environmental excellence and competitiveness, the Port of Long Beach (POLB) plans to improve terminal efficiency by investing in the rail network with the Pier B On-Dock Rail Support Facility Project (Project). To relieve terminal and roadway congestion, future on-dock rail terminal operations will involve movements of small segments of rail cars into and out of supporting yards with greater speed to accommodate long unit trains in the port. The Project includes reconfiguring, expanding, and enhancing the capacity of the existing Pier B Rail Facility, which is primarily located along Pier B Street in the POLB (within the POLB Harbor District). The Project will provide a nearby marshaling area to receive and manage the cargo volume growth and cargo surges, provide a destination for westbound trains that currently are not able to enter the port when on-dock tracks are unavailable, and allow multiple marine terminals to send small cuts of rail cars to be assembled into destination trains. The Project implements critical infrastructure of national significance, which will serve to relieve cargo congestion and prevent bottlenecks such as the current pandemic-related cargo surge at the San Pedro Bay Port Complex.

Most of the Project’s elements are located in the POLB Harbor District (i.e., the POLB’s jurisdiction), however a smaller portion of the Project, including the following elements, are located in existing, dedicated rail right-of-way within the City of Los Angeles (COLA) jurisdiction (see Figure 1.1). Note that some elements are not located in the Coastal Zone.

- Railroad tracks
- Railroad bridge expansion over Dominguez Channel (not in the Coastal Zone)
- Crash walls (under SR-103) (not in the Coastal Zone)
- Track supports (to protected existing structural footings)
- Pavement (access roads)
- Electrical (lighting for rail corridor)
- Compressed air (for train brakes)
- Utility relocations
- Track drainage
- Fire hydrants (at locomotive facility) (not in the Coastal Zone)
- Minor street improvements
2 – Project Permitting and Approvals

An Environmental Impact Report (EIR) encompassing the entire Project was certified by the POLB’s Board of Harbor Commissioners (Board) in January 2018. At the same time, a Coastal Development Permit (CDP), or Harbor Development Permit (HDP), was approved by the Board for the majority portion of the Project located within the Port of Long Beach jurisdiction. At the time that the EIR was under review and certified, the POLB received no comments from the California Coastal Commission.

The POLB is now seeking a Coastal Development Permit for the limited portion of the Project that is located within the COLA, which consists primarily of laying additional railroad track next to existing railroad track in a rail right-of-way, as noted in section 1 above. The HDP that was approved in 2018 was assessed with the best available sea level rise guidance available to the POLB at the time, using a California State Lands Commission-approved climate change adaptation plan (outlined below in section 3), which is the scientific foundation of the POLB’s sea level rise assessment outlined below.

3 – Project Design and Sea Level Rise

Like many transportation-related assets in California, the design life of the Project is anticipated to be approximately 50 years, and the Project has been designed to the highest profile (i.e., greatest height) possible based on connectivity to existing rail and utility infrastructure within and outside of the Project footprint (e.g., surrounding and connecting rail and utility lines,
constraining overhead structures, adjoining roads and freeways, land elevation of neighboring and nearby properties, etc.). The lifespan of the railroad being proposed within the COLA is generally considered to be 40 years; thus, the assumption of a 50-year life span in the Project design analysis is conservative for all Project assets. It should also be noted that the POLB (and other seaports) typically design and redevelop on a roughly 20-30 year planning horizon, based on the nature of its operations and leasing processes, which is a shorter timeframe than other municipalities use along the California coast. Lastly, the design of Project infrastructure located within the City of Los Angeles will need to align with Project design elements in the POLB already approved in January 2018, as noted above in section 2, under the POLB’s CDP/HDP. Given the useful life of the rail lines in the City of Los Angeles and the need for the new rail lines to align with existing rail grade (and the remaining elements of the Project), rail infrastructure will be elevated to the maximum extent possible.

As an integral component of its 2016 Climate Adaptation and Coastal Resiliency Plan (CRP), the POLB prepared a sea level rise hazard and risk analysis for the entire Port of Long Beach Harbor District, which resulted in a suite of seawater inundation maps to be utilized by POLB planners and engineers during design and development permitting processes. These inundation maps, which include the North Harbor area of the POLB (including the Project footprint), can be used to assess POLB construction projects based on the following six sea level rise scenarios per the 2012 National Research Council guidance on sea level rise planning:

- 16” SLR, including a 16” SLR + 100-year storm worst case scenario (approx. year 2050)
- 36” SLR, including a 36” SLR + 100-year storm worst case scenario (approx. year 2070)
- 55” SLR, including a 55” SLR + 100-year storm worst case scenario (approx. year 2100)

The Project’s Structural Analysis Report (see Appendix A) includes a sea level rise and overtopping assessment at all six of these scenarios on all proposed railroad and supporting infrastructure associated with the Project. Potential seawater inundation on Project components is shown in feet on each major asset outlined in the report.

Based on the anticipated 50-year design lifespan in the Project’s design analysis, the objective of this report is to show that the year 2070 sea level rise scenario (i.e., the 36” SLR + 100-year storm worst case scenario) analyzed in the POLB’s CRP is consistent with the best available science and guidance at the time the CDP/HDP was issued and remains in alignment with the California Coastal Commission Sea Level Rise Policy Guidance originally adopted in August 2015 and updated in November 2018 to incorporate sea level rise projections released by the California Ocean Protection Council (OPC) in 2018. More specifically, the table below outlines how the CRP’s year 2070 scenario is in alignment with the 2018 OPC guidance for the year 2070 planning horizon in the Los Angeles region (per table 28 of the 2018 OPC guidance report) at both the low and medium-high risk aversion scenarios. As the Project’s design life is 50 years, the year 2070 planning horizon in the 2018 OPC guidance is the most appropriate to use for basing the analysis of potential future sea level rise on Project assets. The POLB also intends to use the CRP’s existing 55” SLR + 100-year storm scenario (i.e., the CRP’s “end-of-century, worst-
case scenario”) to align well with the H++ extreme risk aversion scenario at year 2070 in the 2018 OPC guidance.

Table 3-1 – Comparison of POLB CRP Sea Level Rise at 2070 versus 2018 OPC Guidance at 2070 for the Los Angeles Region at Low, Medium-High, and Extreme Risk Scenarios

<table>
<thead>
<tr>
<th></th>
<th>Low Risk Aversion Scenario</th>
<th>Medium-High Risk Aversion Scenario (1 in 200 chance)</th>
<th>H++ Extreme Risk Aversion Scenario</th>
</tr>
</thead>
<tbody>
<tr>
<td>OPC Low Emissions 2070</td>
<td>1.3 feet</td>
<td>2.9 feet</td>
<td>5.0 feet</td>
</tr>
<tr>
<td>OPC High Emissions 2070</td>
<td>1.7 feet</td>
<td>3.3 feet</td>
<td></td>
</tr>
<tr>
<td>POLB CRP 2070</td>
<td>1.3 feet</td>
<td>3 feet</td>
<td>4.5 feet**</td>
</tr>
</tbody>
</table>

**The POLB CRP does not assess a H++ extreme risk aversion scenario, as that scenario was not available at the time of CRP development; however, the CRP’s 55” SLR + 100-year storm scenario (4.5 ft. of rise) aligns well with the H++ scenario at year 2070 (5 ft. of rise).

It should be noted that, per these various scenarios, there is a potential for seawater inundation at the year 2070 planning horizon on most Project assets. However, as noted above, the Project has been designed to the highest profile possible based on surrounding rail and utility infrastructure within and outside of the Project footprint (e.g., surrounding and connecting rail and utility lines, constraining overhead structures, adjoining roads and freeways, land elevation of neighboring and nearby properties, etc.). Whether the POLB assesses the Project with the 2018 OPC guidance’s medium-high risk aversion scenario or the POLB CRP’s year 2070 scenario (i.e., the 36” + 100-year scenario), this inundation will still be likely, and any potential adaptation strategies would not change based on the assessment. Similarly, if the POLB were to assess the Project with the 2018 OPC guidance’s H++ extreme risk aversion scenario or the POLB CRP’s year 2100 scenario (i.e., the 55” + 100-year worst-case scenario), many Project assets would be inundated regardless and there would be no change in potential mitigation strategies for Project assets.

Forecasting out beyond year 2070 to the 2100 end-of-century scenarios included in the 2018 OPC guidance is inappropriate for this analysis, as the Project will likely have already met its end-of-life, and the POLB will likely have redeveloped the Pier B/North Harbor region long before the end of the century. Additionally, while important for long-term coastal risk management, particularly in densely populated coastal zones, high-end and end-of-century scenarios are somewhat speculative. Thus, the POLB concluded that additional sea level rise modeling and mapping specific to this Project with the 2018 OPC guidance is not necessary. As the POLB typically designs and redevelops on a 20-30 year planning horizon (depending on the type and function of assets), which is shorter than a typical coastal municipality would redevelop its assets over time, the planning scenarios outlined above are appropriate to apply to the Project. However, with the relatively rapid advancement of climate change science, future sea level rise models could be used to assess potential future impacts on other large-
scale POLB development projects, and the POLB is currently working to update the POLB CRP’s port-wide sea level rise inundation maps based on the most current 2018 OPC guidance within the next year. Note that the newest sea level rise guidance was not yet available to State grantees at the time of CDP/HDP approval. POLB management and/or adaptation strategies, opportunities, and priorities are not expected to significantly change, if at all, based on newer modeling though.

4 – Potential Sea Level Rise Impacts

The Project includes many elements that would be considered critical assets per the California Coastal Commission’s (CCC) recent draft *Critical Infrastructure at Risk: Sea Level Rise Planning Guidance for California’s Coastal Zone*, including highways, roads, railroads, wastewater, stormwater, and water supply infrastructure. The Project’s Structural Analysis Report (see Appendix A) includes a sea level rise and overtopping assessment at all six CRP sea level rise scenarios on the following Project design elements (see section 1), which are located within existing, dedicated rail right-of-way within the City of Los Angeles jurisdiction (see Figure 4-1):

- Railroad tracks
- Railroad bridge expansion over Dominguez Channel (not in the Coastal Zone)
- Crash walls (under SR-103) (not in the Coastal Zone)
- Track supports (to protected existing structural footings)
- Pavement (access roads)
- Electrical (lighting for rail corridor)
- Compressed air (for train brakes)
- Utility relocations
- Track drainage
- Fire hydrants (at locomotive facility) (not in the Coastal Zone)
- Minor street improvements
5 – Infrastructure Adaptation Approach

Climate resilience measures have already been incorporated into the Project design elements within the COLA jurisdiction. Mechanical (e.g., compressed air) and electrical (e.g., electricity for rail corridor) design elements have been elevated (or can be easily elevated) above existing grade to reduce potential flooding impacts in coming decades. Track drainage and associated pumping systems capacity have been designed to accommodate additional drainage needs resulting from increased precipitation or coastal inundation. Rail infrastructure will also be elevated to the maximum extent possible, with constraints of existing connected or overhead structures, to reduce any flooding impacts. Railroad tracks and street pavement, however, are naturally resilient elements as inundation would only result in temporary downtime and a rapid regain in operations once water levels recede.

Existing right of way within the City of Los Angeles jurisdiction is extremely limited to implement additional large-scale resilience measures. However, additional intermediate and end-of-lifespan resilience measures could be incorporated into the Project should sea level rise inundation become a frequent concern in the coming decades. Design elements like the inclusion of sea walls, embankments, or additional drainage pathways could help to reduce future inundation frequency. Changes in facility usage in port districts often modulate the scope and scale of facility drainage pathways. Ideally, these small-scale features are added once facility operators have identified the flooding impacts to the facility operations and components, and subsequently design the features to fit within the existing facility.
Additionally, the inclusion of inundation-specific emergency procedures and training with dedicated and trained staff could reduce the downtime of the Project operations in the future.

6 – References


POLB (Port of Long Beach). 2016a. Port of Long Beach Climate Adaptation and Coastal Resiliency Plan.
ATTACHMENT 5
POLB Profile Technical Memo
PROFILE
February 15, 2022

CCC Comments:
1. Need definition of what “highest profile” is in relation to the inundation.
2. Describe, in greater details, the physical constraints which limit the profile of the improvements. Discuss why it will be infeasible to raise the profile any further.
3. Be more specific about impacts related to elevating the profile further:
   a. Will it be too costly to further raise the profile?
   b. Will it be conflicting with the existing infrastructure?

Pier B ODRSF Program track and surrounding grade elevations have been maximized to their highest practical limit given key vertical constraints in the surrounding area while meeting state clearance requirements and following established railroad design practices. California Public Utilities Commission (CPUC) General Order 26 defines the minimum vertical clearance between the top of rail and obstructions as 22’-6”. Track elevations have been set at this minimum clearance beneath the key existing overhead constraints. In addition, the American Railway Engineering and Maintenance-of-Way Association (AREMA) has established guidance for track profile grades which limit the Program’s ability to raise grades in between these constraints.

Figure 1 shows five constraints that make further raising the track and grade infeasible.

![Figure 1: Main Vertical Constraints](image)

The key constraints are also described below:

1. Dominguez Channel Bridge – The existing rail bridge is a vital link from the POLB to the Alameda Corridor. Bridge and neighboring track reconstruction including the Long Beach Lead Track and Alameda Corridor Transportation Authority (ACTA) Lead Tracks to the east and west, the San Pedro Branch Track to the north and the Terminal Island Lead Tracks to the south would be required to significantly raise the track profile. Any track raises in this area would impact private properties to the north and south of the existing rail corridor
due to the tight right-of-way constraints in the area. Proposed track elevations in this area are approximately 19.5 FT mean lower low water (MLLW).

2. SR-103 Bridge Soffit – The track profile grades have been maximized in this area while still meeting state and federal clearance requirements. Raising tracks further would require full reconstruction of the bridge. Proposed track elevations in this area are approximately 9.5 FT MLLW.

3. Anaheim Street Bridge Soffit – The track profile grades have been maximized in this area while still meeting state and federal clearance requirements. Raising tracks further would require full reconstruction of the bridge. Proposed track elevations in this area are approximately 5.3 FT MLLW.

4. Spur Track Access to Facilities – The track profiles must be able to match the grades entering the facilities off Pier B Street. Raising tracks in this area would require full reconstruction of the facilities. Proposed track elevations in this area are approximately 10.3 FT MLLW.

5. SR-710 Bridge Soffit – The track profile grades have been maximized in this area while still meeting state and federal clearance requirements. Raising tracks further would require full reconstruction of the freeway off-ramp bridge structure. Proposed track elevations in this area are approximately 12.6 FT MLLW.

Removing any of these constraints is not viable as it will involve multiple jurisdictions at local, regional, state, and federal levels, require very high amount of capital investments, and result in unreasonable real property impacts.
ATTACHMENT 6
Final Structural Analysis
Final Structural Analysis Report
Pier B On-Dock Rail Support Facility Program

January 2022
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<table>
<thead>
<tr>
<th>Acronym</th>
<th>Description</th>
</tr>
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<tbody>
<tr>
<td>3D</td>
<td>three-dimensional</td>
</tr>
<tr>
<td>AASHTO</td>
<td>American Association of State Highway and Transportation Officials</td>
</tr>
<tr>
<td>ACI</td>
<td>American Concrete Institute</td>
</tr>
<tr>
<td>ACTA</td>
<td>Alameda Corridor Transportation Authority</td>
</tr>
<tr>
<td>ADA</td>
<td>Americans with Disabilities Act</td>
</tr>
<tr>
<td>AISC</td>
<td>American Institute of Steel Construction</td>
</tr>
<tr>
<td>AREMA</td>
<td>American Railway Engineering and Maintenance-of-Way Association</td>
</tr>
<tr>
<td>ARS</td>
<td>acceleration response spectra</td>
</tr>
<tr>
<td>ASCE</td>
<td>American Society of Civil Engineers</td>
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<tr>
<td>AWS</td>
<td>American Welding Society</td>
</tr>
<tr>
<td>BDP</td>
<td>Bridge Design Practice by Caltrans</td>
</tr>
<tr>
<td>bgs</td>
<td>below ground surface</td>
</tr>
<tr>
<td>CalOSHA</td>
<td>California Occupational Safety and Health Administration</td>
</tr>
<tr>
<td>Caltrans</td>
<td>California Department of Transportation</td>
</tr>
<tr>
<td>CBC</td>
<td>California Building Code</td>
</tr>
<tr>
<td>CCC</td>
<td>California Coastal Commission</td>
</tr>
<tr>
<td>CEC</td>
<td>California Electrical Code</td>
</tr>
<tr>
<td>CEN</td>
<td>California Energy Code</td>
</tr>
<tr>
<td>CFC</td>
<td>California Fire Code</td>
</tr>
<tr>
<td>CGB</td>
<td>California Green Building</td>
</tr>
<tr>
<td>CIDH</td>
<td>cast-in-drill-hole</td>
</tr>
<tr>
<td>CIP</td>
<td>cast-in-place</td>
</tr>
<tr>
<td>CISS</td>
<td>cast-in-steel shell</td>
</tr>
<tr>
<td>CLE</td>
<td>contingency level earthquake</td>
</tr>
<tr>
<td>CMC</td>
<td>California Mechanical Code</td>
</tr>
<tr>
<td>COLA</td>
<td>City of Los Angeles</td>
</tr>
<tr>
<td>COLB</td>
<td>City of Long Beach</td>
</tr>
<tr>
<td>CRP</td>
<td>Coastal Resiliency Plan</td>
</tr>
<tr>
<td>DCR</td>
<td>demand to capacity ratio</td>
</tr>
<tr>
<td>DIB</td>
<td>Design Information Bulletin</td>
</tr>
<tr>
<td>Dominguez Channel Bridge</td>
<td>Long Beach Lead Railroad Bridge over Dominguez Channel</td>
</tr>
<tr>
<td>DYA</td>
<td>Diaz Yourman Associates</td>
</tr>
<tr>
<td>EFP</td>
<td>equivalent fluid pressure</td>
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<tr>
<td>EI</td>
<td>Elevation</td>
</tr>
<tr>
<td>EMI</td>
<td>Earth Mechanics, Inc.</td>
</tr>
<tr>
<td>EQ</td>
<td>earthquake</td>
</tr>
<tr>
<td>FCM</td>
<td>fracture critical members</td>
</tr>
<tr>
<td>FHWA</td>
<td>Federal Highway Administration</td>
</tr>
<tr>
<td>ft²</td>
<td>square feet</td>
</tr>
<tr>
<td>GDB</td>
<td>Gerald Desmond Bridge</td>
</tr>
<tr>
<td>k/ft</td>
<td>kilopound(s) per foot/feet</td>
</tr>
<tr>
<td>Kd</td>
<td>directionality factor</td>
</tr>
<tr>
<td>Abbreviation</td>
<td>Description</td>
</tr>
<tr>
<td>--------------</td>
<td>-------------</td>
</tr>
<tr>
<td>Kz</td>
<td>Velocity Pressure Exposure Coefficient</td>
</tr>
<tr>
<td>Kzt</td>
<td>topographic factor</td>
</tr>
<tr>
<td>LA</td>
<td>Los Angeles</td>
</tr>
<tr>
<td>lb/ft</td>
<td>pound(s) per foot/feet</td>
</tr>
<tr>
<td>lb(s)</td>
<td>pound(s)</td>
</tr>
<tr>
<td>LADPW</td>
<td>Los Angeles Department of Water and Power</td>
</tr>
<tr>
<td>LED</td>
<td>light-emitting diode</td>
</tr>
<tr>
<td>LFD</td>
<td>Load Factor Design</td>
</tr>
<tr>
<td>LRFD</td>
<td>Load Resistance Factor Design</td>
</tr>
<tr>
<td>m/s</td>
<td>meters per second</td>
</tr>
<tr>
<td>MEP</td>
<td>Mechanical/Electrical/Plumbing</td>
</tr>
<tr>
<td>MPH</td>
<td>mile(s) per hour</td>
</tr>
<tr>
<td>MSE</td>
<td>Mechanical Stabilized Embankment</td>
</tr>
<tr>
<td>NAVD88</td>
<td>North American Vertical Datum of 1988</td>
</tr>
<tr>
<td>NFPA</td>
<td>National Fire Protection Association</td>
</tr>
<tr>
<td>OLE</td>
<td>operating level earthquake</td>
</tr>
<tr>
<td>pcf</td>
<td>pounds per cubic feet</td>
</tr>
<tr>
<td>POLA</td>
<td>Port of Los Angeles</td>
</tr>
<tr>
<td>POLB</td>
<td>Port of Long Beach</td>
</tr>
<tr>
<td>Program</td>
<td>Pier B On-Dock Rail Support Facility Program</td>
</tr>
<tr>
<td>PS&amp;E</td>
<td>Plans, Specifications, and Estimate</td>
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<tr>
<td>psf</td>
<td>pounds per square foot</td>
</tr>
<tr>
<td>PSI</td>
<td>pounds per square inch</td>
</tr>
<tr>
<td>q_e</td>
<td>Velocity Pressure</td>
</tr>
<tr>
<td>RWQCB</td>
<td>Regional Water Quality Control Board</td>
</tr>
<tr>
<td>SCIG</td>
<td>Southern California International Gateway</td>
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<td>SCRRA</td>
<td>Southern California Regional Rail Authority</td>
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<tr>
<td>SDC</td>
<td>Seismic Design Criteria by Caltrans</td>
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<td>SLD</td>
<td>Service Load Design</td>
</tr>
<tr>
<td>SLR</td>
<td>sea level rise</td>
</tr>
<tr>
<td>SR-103</td>
<td>State Route 103</td>
</tr>
<tr>
<td>SR-710</td>
<td>State Route 710</td>
</tr>
<tr>
<td>STP</td>
<td>Structural Technical Policies</td>
</tr>
<tr>
<td>TMS</td>
<td>The Masonry Society</td>
</tr>
<tr>
<td>USACE</td>
<td>United States Army Corps of Engineers</td>
</tr>
<tr>
<td>USGS</td>
<td>United States Geological Survey</td>
</tr>
<tr>
<td>V</td>
<td>Version</td>
</tr>
<tr>
<td>V</td>
<td>wind speed</td>
</tr>
<tr>
<td>V&lt;sub&gt;530&lt;/sub&gt;</td>
<td>average shear wave velocity within the upper 30 meters</td>
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</tbody>
</table>
1.0 Introduction

1.1 Project Background

To maintain its environmental excellence and competitiveness, the Port of Long Beach (POLB) plans to improve terminal efficiency by investing in the rail network. To relieve terminal and roadway congestion, future on-dock rail terminal operations will involve movements of small cuts of rail cars into and out of supporting yards with greater speed to accommodate long unit trains in the port. The Pier B On-Dock Rail Support Facility Program (Program) includes reconfiguring, expanding, and enhancing the capacity of the existing Pier B Rail Facility, which is located along Pier B Street in the port. The Program will provide a nearby marshaling area to receive and manage the cargo volume growth and cargo surges, provide a destination for westbound trains that currently are not able to enter the port when on-dock track space is unavailable, and allow multiple marine terminals to send small cuts of rail cars to be assembled into destination trains. Figure 1-1 depicts a conceptual rendering of the Program once completed.

Figure 1-1. Conceptual Pier B On-Dock Rail Support Facility Program Rendering
The Program proposes to:

- Relocate existing mainline tracks into the North Harbor area.
- Reconfigure existing tracks and add additional tracks to allow five arrival/departure tracks up to 10,000 feet long with direct connection to the on-dock rail facilities and the Alameda Corridor railway totaling 52,000 additional track-feet.
- Provide for additional rail car storage and staging with 38 storage tracks totaling 93,000 track-feet.
- Construct locomotive storage and fueling facility.
- Relocate three at-grade railroad crossings on Anaheim Way and Pier B Street.
- Realign Pico Avenue to provide clearance for the rail corridor.
- Widen and realign Pier B Street to improve safety and truck traffic.
- Provide efficient street lighting and rail yard lighting.
- Partially remove the Shoemaker Bridge ramps.
- Remove existing streets between 9th Street and 12th Street in the North Harbor area.
- Widen existing Dominguez Channel Bridge to accommodate additional track.
- Provide a crash wall to protect existing State Route 103 (SR-103) (Terminal Island Freeway) and Anaheim Street overcrossing bridge columns.
- Construct retaining wall and tie-back wall along State Route 710 (SR-710) to accommodate eight railroad tracks.
- Modify existing Berth 54 Crescent Warehouse to accommodate Pico Avenue realignment.
- Construct a storm drain and sewer lift station.
- Demolish the existing Los Angeles County LA-04 Pump Station and construct a pump station at a new location.
- Construct Sewer Lift Station
- Relocate public utilities outside of the rail corridors.
- Coordinate third-party utility relocations outside of the rail corridors.
- Relocate the existing 16 active oil-production-associated wells outside of the rail corridors.

See Figure 1-2 for the Pier B On-Dock Rail Support Facility Program Site Plan.
1.2 Report Background

A diverse set of buildings, bridges, walls, vaults, and other structural elements exist within, above, and adjacent to the project footprint. These structural features serve functions ranging from direct support of the Pier B rail yard functions to aerially spanning the yard with public roadways. The Program involves the design of new structures and modification to existing structures, including the widening of an existing rail bridge over the Dominguez Channel, retaining and crash walls to protect existing infrastructure, realignment of Pico Avenue and 9th Street, modification of the Crescent Warehouse building, and miscellaneous other structures and buildings.

This report is provided as an initial step in the development of structural design for the project’s structures and will serve as a reference document to provide guidance to the structural engineering team. Recommendations are provided that will be integrated with the design of other disciplines and may potentially impact the overall design approach.
2.0 Purpose

The objectives of this report are to:

A. Establish design criteria associated with each structure in accordance with the agency having jurisdiction, and identify applicable codes, assumptions, and constraints associated with the design of each structure

B. Identify potential structural interaction with existing structures and utilities

C. Perform structural analysis to determine the internal forces for each structure under service, determine wind and seismic loading, size the main structural elements, and identify the structural performance against seismic demand
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3.0 Approach

To meet the purpose of this study, the following approach was taken:

- **Establish Design Criteria:** Provided a comparison summary of alternatives for each structure identified below in compliance with the agency having jurisdiction and applicable codes. Refer to Figure 3-1 for each structure location.
  A. Long Beach Lead Railroad Bridge over Dominguez Channel
  B. SR-103 Crash Walls
  C. Anaheim Street Overcrossing Tracks Support
  D. Anaheim Street Overcrossing Retaining Wall
  E. Pier B Street Retaining Wall
  F. LA-04 Pump Station
  G. Sewer Lift Station
  H. SR-710 Retaining Wall
  I. Crescent Warehouse Alteration

- **Identify Potential Conflict for Existing Structures:** Considered existing structures/utilities and their potential interaction with the new structures; incorporated field data and geotechnical information to assist in identifying potential conflicts, complicated elements, or extensive utility coordination associated with the work.

- **Structural Alternative Analyses:** Performed structural analyses to verify and compare structural performance of various alternatives.
4.0 Long Beach Lead Railroad Bridge over Dominguez Channel

The existing Long Beach Lead Railroad Bridge over Dominguez Channel (designated herein as the Dominguez Channel Bridge) is located west of the Pier B Rail Yard (refer to Figure 3-1 for the bridge location and Figure 4-1 for bridge overview). The existing Dominguez Channel Bridge accommodates two Alameda Corridor Transportation Authority (ACTA) mainline railroad tracks that connect POLB’s Pier B Rail Yard and adjacent on-dock rail terminals to the Alameda Corridor.

*Figure 4-1. Existing Dominguez Channel Bridge Overview*

The existing Dominguez Channel Bridge was built circa 2001. The bridge structure is 204 feet long and 38 feet wide, carrying two railroad tracks and two 2-foot, 6-inch-wide walkways. Several existing utility lines are located near the existing Dominguez Channel Bridge; see Section 4.2.1 for details.

Additional rail track is proposed to be added south of the existing Dominguez Channel Bridge. To accommodate the proposed track, the existing bridge needs to be widened. The purpose of this
study is to investigate the potential addition of a third track to the Dominguez Channel Bridge. It is assumed that the proposed third track would be located either north or south of the existing ACTA main tracks at 15-foot track centers from the adjacent track. The bridge structure required to support the three tracks would be approximately 53 feet wide. It is also assumed that the initial build-out configuration of the Pier B Rail Facility would leave the ACTA main tracks on their current alignment across the existing bridge.

A site visit was performed on December 12, 2019, to investigate the bridge area and identify above-ground constraints and potential conflicts. Potential site constraints are identified in the Pier B Site Investigation Report 1-2-3 (HDR, 2019).

4.1 Basis of Design

4.1.1 Applicable Codes, Standards, and Guidelines

The structures will be designed and constructed in accordance with the latest version of the following codes and standards:

- California Department of Transportation (Caltrans) Bridge Design Practice (BDP) (Caltrans, 2015)
- Caltrans Seismic Design Criteria (SDC), Version (V) 2.0 (Caltrans, 2019)
- Caltrans Structural Technical Policies (STPs)
- Los Angeles Department of Water and Power (LADPW) Policy and Standards
- U.S. Army and Coast Guard Policy

4.1.2 Materials

- Concrete: minimum compressive strength at 28 days for cast-in-place (CIP) reinforced concrete abutments, piers, foundations, and cast-in-steel shell (CISS) pile in-fill = 4,000 pounds per square inch (PSI)
- Reinforcing steel: Will conform to the requirements of ASTM A706, Grade 60; all spiral reinforcement will conform to the requirements of ASTM A1064
- Structural steel: Will conform to the requirements of ASTM A709, Grade 50W; handrail steel will conform to the requirements of ASTM A500, Grade B
- Steel members and components shown as fracture critical members (FCM): Will be designed in accordance with AREMA (2019) Chapter 15, Section 1.14
- Bolts: Will conform to the requirements of ASTM F3125 Type, Class B Slip Critical, high strength
- Anchor bolts: Will conform to the requirements of ASTM A354, Grade BC, and will be hot dipped galvanized after fabrication

4.1.3 Loads and Load Combinations

Loads
- Materials Unit Weight:
  - Concrete = 150 pounds per cubic feet (pcf)
  - Steel = 490 pcf
  - Rail ballast = 120 pcf
- Added Dead Load: 24-inch rail ballast and utilities
- Track Load: 200 pounds per feet (lb/ft) per track
- Live Load: Cooper E80 live load per AREMA (2019) Volume 2, Chapter 15, Part 1, Section 1.3.3
- Impact: AREMA (2019) Chapter 8, Part 2 for concrete structure; Chapter 15, Part 1, Section 1.3.5 for steel structures
- Design Speed: 25 miles per hour (MPH), per existing bridge design speed
- Utilities: 150 pounds per square foot (psf) existing; to be confirmed for the final design of the new widening with the utilities team
- Temperature Load: Total temperature differential of 70 degrees Fahrenheit per AREMA (2019) Section 2.2.3
- Wind Load: Wind loading for both loaded and unloaded condition per AREMA (2019) Chapter 15, Part 1, Sections 15.1.3.7 and 15.1.3.8; due to the large dead load on the structure and high seismicity at the bridge site, wind loading is not expected to govern design
- *Tsunami Hazard Assessment Report* (M&N, 2007) stated that for the bulk of the area within the POLB, no water overtopping is expected; therefore, tsunami loads are not considered in the design
- Seismic Mass: Dead load and added dead load
- Seismic Load: Three level earthquakes (EQs) per AREMA (2019) Chapter 9, Part 1.3.2, Table 9-1-4.
Per AREMA seismic criteria, the acceptable risk criteria with respect to Level 1 ground motion will consider the safety and continuing operation of trains with speed restrictions. For Levels 2 and 3 ground motion, the acceptable risk criteria will be based mainly on economic considerations unless the bridge has a high passenger train occupancy rate. Train traffic is stopped per Railroad Response Level III for Levels 2 and 3 ground motions until bridge inspections are completed.

The structure importance classification system is used to determine the appropriate average ground motion return period for each of the following limit states: serviceability, ultimate, and survivability. The importance of a structure is determined by three measures: Immediate Safety, Immediate Value, and Replacement Value. These measures are combined in AREMA (2019) Chapter 9, Part 1, Section 1.3.2.2.4, to determine the appropriate return period for each of the limit states. Table 4-1 provides the factors used for the Dominguez Channel Bridge calculation.

### Table 4-1. Bridge Importance Measure Factors

<table>
<thead>
<tr>
<th>Importance Measure</th>
<th>Factor Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Immediate Safety</td>
<td>4</td>
</tr>
<tr>
<td>Immediate Value</td>
<td>4</td>
</tr>
<tr>
<td>Replacement Value</td>
<td>2.25</td>
</tr>
</tbody>
</table>

The proposed bridge widening is connected to the existing structure; therefore, the combined bridge, existing and widening, will be designed for the estimated three level EQs per AREMA (2019). Based on the results provided in Table 4-1, the new structure will be designed for the Levels 1, 2, and 3 EQs for 100-, 463- and 1,959-year return periods, respectively.

Per AREMA (2019), during Level 1 ground motion, the critical bridge members will remain in the elastic range. During Levels 2 and 3, the structure may respond beyond the elastic range and allow structural damage without bridge collapse. Table 4-2 summarizes the return period and performance criteria for each seismic level for the Dominguez Channel Bridge. AREMA (2019) does not include ductility for local or global bridge columns limits; however, it does recommend special detailing as well as reinforcing and confinement requirements.

To ensure the widened bridge will perform in ductile behavior under an extreme seismic event, Caltrans SDC V 2.0 (2019) and Caltrans STPs will be followed. Per STP 20-3, which addresses the SDC for bridge widenings, the seismic evaluation of the existing structure will conform to STP 16.7, while the newly widened portions of the bridge will comply with Caltrans SDC V 2.0, except portions related to balanced stiffness criteria (SDC V 2.0, Section 7.1 [Caltrans, 2019]).
### Table 4-2. AREMA Performance Criteria

<table>
<thead>
<tr>
<th>Limit State</th>
<th>Return Period (years)</th>
<th>Performance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Serviceability</td>
<td>100</td>
<td>Elastic/No Damage</td>
</tr>
<tr>
<td>Ultimate</td>
<td>463</td>
<td>Accessible and Repairable Damage</td>
</tr>
<tr>
<td>Survivability</td>
<td>1,959</td>
<td>Significant Damage/No Collapse</td>
</tr>
</tbody>
</table>

### Load Combinations

The recommended load combinations will be based on AREMA (2019) as follows:

- Chapter 8, Section 2.2.4, Table 8-2-4, Service Load Design (SLD):
  - Structural steel, piles, and footings
- Chapter 8, Section 2.2.4, Table 8-2-5, Load Factor Design (LFD):
  - Concrete abutments and bent caps; bent caps will be designed for plastic hinging in the columns
- For earthquake load combination, 25 percent of live load is included
- For earthquake loading, bi-directional inertial loading will be considered; additionally, 50 percent inertial + 100 percent kinematic will be considered
- Caltrans BDP (2015) Chapter 16, Table 16.1-3:
  - 100 percent scour will be combined with Level 1 ground motion

### 4.1.4 Serviceability

#### Service Life

AREMA does not clearly specify the recommended design life for railroad bridges. However, AASHTO specifies bridges design lift as 75 years. AASHTO design life definition has been implemented in this analysis.

#### Deflection

Per AREMA (2019) Chapter 15, Section 1.2.5, members will be designed so vertical deflection due to live load plus impact does not exceed 1/640 of the span.

#### Fatigue

Structural elements will be designed for fatigue per AREMA (2019) Chapter 15, Section 1.3.13.
Paint System

Structural steel will be coated with epoxy-based paint, conforming to the requirements of AREMA (2019) Chapter 15, Section 8.7. The consideration of weather-resistant steel might be considered in the Final Plans, Specifications, and Estimate (PS&E) stage.

Corrosion

For existing bridge and proposed widening, sacrificial steel thickness of 1/8 inch is considered for design purposes per Caltrans corrosion guidelines.

Waterproofing

All waterproofing membranes will conform to AREMA (2019) Chapter 8, Section 29.2.

4.1.5 Sea Level Rise

The Climate Adaptation and Coastal Resiliency Plan (CRP) prepared by POLB (2016) includes three scenarios for overtopping and inundation, with and without surge, for the 100-year return period. Table 4-3 summarizes the impact of sea level rise (SLR) for these scenarios at the Dominguez Channel Bridge. The SLR is not considered in the design as agreed upon with POLB.

<table>
<thead>
<tr>
<th>Table 4-3. Sea Level Rise Impact at Dominguez Channel Bridge</th>
</tr>
</thead>
<tbody>
<tr>
<td>SLR Scenario</td>
</tr>
<tr>
<td>----------------</td>
</tr>
<tr>
<td>100-year Storm Surge Scenario Included</td>
</tr>
<tr>
<td>Dominguez Channel Railroad Bridge Inundation</td>
</tr>
</tbody>
</table>

*a Indicates no potential water accumulation

4.2 Existing Utilities and Right-of-Way Impact

4.2.1 Utilities and Potential Conflicts

Per as-built drawings, there is an existing 12-inch DWP oil line and pipe support along the south side of the bridge. Approximately 20 feet to the south is a utility bridge that crosses the Dominguez Channel and supports numerous petroleum-related utility lines. Approximately 50 feet to the north is another utility bridge that carries a 12-inch diameter Arco oil line and an 8-inch GATX oil line. Existing electric overhead lines cross over the railroad tracks from the northeast corner to the
southwest corner of the bridge. Six 4-inch conduits are located within the ballast at the south side of the bridge. The utility plan for the Dominguez Channel Bridge area is provided in Figure 4-2.

**Figure 4-2. Dominguez Channel Bridge Utilities Plan**

During the site visit on December 12, 2019, additional above-ground conflicts were identified (see *Pier B Site Investigation Report 1-2-3* [HDR, 2019]). To accommodate the proposed widening, the existing 12-inch DWP oil line along the south side of the bridge will be relocated along the widened bridge. Additionally, the electric overhead lines crossing over the railroad tracks from the northeast corner to the southwest corner of the bridge may need to be temporarily relocated to allow for the installation of the CISS piles and steel girders. Detailed existing utilities exhibits are shown in Appendix A.

### 4.2.2 Right-of-Way Impact

The widened bridge would encroach on the right-of-way limit at the south side of the bridge by approximately 7 feet (see Figure 4-8), which would require acquisition of right-of-way from the City of Los Angeles (COLA) and Los Angeles (LA) County Flood Control District. Also, the Dominguez Channel Bridge is under ACTA jurisdiction.
4.3 Geotechnical Considerations

Preliminary geotechnical design recommendations are provided in the *Geotechnical Memorandum* prepared by HDR (2020); see Appendix B for details.

4.3.1 Subsurface Conditions

Based on available information from historical borings, subsurface materials at the site generally consist of natural deposits of silty or clayey sand to a depth of approximately 15 feet below ground surface (bgs), to approximate elevation (El) 0 feet, North American Vertical Datum of 1988 (NAVD88; all elevations use this datum). Soft to hard clays and silts were encountered below this zone to approximate El -20 feet, NAVD88. Beneath this layer, soils were generally medium dense to very dense sandy materials, with density increasing with depth. Isolated layers of soft, fine-grained materials were noted in two borings within this zone (around El -50 feet, NAVD88, at G-131 BR-2; and El -65 feet, NAVD88, at EMI-MG-24). Groundwater was noted in two borings and is expected to be present at approximate El +5 feet, NAVD88.

Measured dry density of the fine material between depths of approximately 20 and 40 feet bgs ranged between 76.4 and 93.3 pcf, with an average of 83.8 pcf. One density test was shown on the boring logs at a depth of approximately 75 feet bgs within the granular material, which indicated a dry density of 104.7 pcf.

4.3.2 Liquefaction and Lateral Spreading

The potential for liquefaction at the project site exists due to expected shallow groundwater depths. Additionally, the project site is located within an area designated as potentially liquefiable by the California Geological Survey (1999). Liquefaction-induced settlements are estimated to range from approximately 2 to 10 inches within the project site (Leighton and Associates, 1996; EMI, 2000). Preliminary calculations estimate settlements to be up to 11 inches.

Updated ground motions may differ from those used in previous studies, and additional subsurface data will be obtained for this Program to be used in future liquefaction evaluations.

Earth Mechanics Inc. (EMI, 2000) estimated that liquefaction potential exists for ground motions corresponding to return periods of 72 and 1,000 years. Current design ground motions are expected to be at least as high as EMI’s 72-year ground motions. Therefore, it is assumed that liquefaction potential exists for all current ground motions. These evaluations will be updated and refined during final design following completion of the geotechnical investigation.

Potential for lateral spreading exists at the site due to factors such as relatively steep slopes, shallow groundwater, liquefiable soils, and soft clays. A lateral spreading analysis will be performed that considers pile layout, sizing, slope stability modeling, pile-soil interaction, and
design ground motions. EMI (2000) considered various scenarios that may or may not be applicable to the proposed structures; however, they only apply to the ground motions identified in the report. The application of lateral spreading load and its effects on proposed foundations should be anticipated.

4.3.3 Scour

The potential for scour will be considered in design of the pier foundations as they are located within the channel. For preliminary design purposes, a full scour depth of 12 feet was assumed at all piers based on as-built plans for the previous widening of the bridge.

4.3.4 Corrosion

Based on experience, fine-grained soils (sils and clays) are typical soil types responsible for corrosive site conditions. Due to the predominance of fine-grained foundation soils anticipated within the pile depths, corrosive soils should be anticipated. Additionally, salt water is anticipated on site. For the existing bridge, EMI (2000) recommended a corrosion rate of 0.0025 inch per year according to Caltrans corrosion guidelines for steel piles with a sacrificial steel thickness of 1/8 inch. Corrosion protection measures, including sacrificial steel thickness, will be considered in final design.

4.3.5 Preliminary Geotechnical Recommendations

Seismic Design Spectra

Acceleration response spectra (ARS) corresponding to the three seismic levels indicated in AREMA (2019) will be used for design of the proposed bridge. These seismic events include Level 1 (100-year return period), Level 2 (463-year return period), and Level 3 (1,959-year return period).

At each return period, HDR developed ARS curves for two boundary site classes (C/D and D/E) using the U.S. Geological Survey’s (USGS, 2020) unified hazard tool. Boundary of Site Class C/D with an average shear wave velocity within the upper 30 meters ($V_{s30}$) of 360 meters per second (m/s) was representative of a firm ground, resulting in higher spectral accelerations at lower spectral periods. Boundary of Site Class D/E with a $V_{s30}$ of 180 m/s was selected for the soft ground, resulting in higher spectral accelerations at higher periods. The preliminary ARS for the site was selected as the envelope of these two ARS curves increased by 20 percent to account for softer than assumed soil profiles and the impact from the Wilmington Fault that was not considered in the USGS fault models. The preliminary ARS curves for the return periods of 100, 463, and 1,959 years are shown in Figure 4-3. The ARS curves will be re-evaluated after completion of the geotechnical investigation.
Lateral Earth Pressure

Appendix B provides a set of equivalent fluid pressure (EFP) values for the preliminary design of earth-retaining structures at the site. The EFP concept is commonly used in the estimation of lateral earth pressure that a retaining wall or shoring system will be required to resist. EFP is expressed as the unit weight of a fluid (in pcf) that would generate a hydrostatic pressure equal to the anticipated lateral earth pressure at a given depth. This horizontal pressure is applied to a vertical plane extending up from the heel of the wall base, and the weight of soil above the wall heel is included as part of the wall weight. A soil unit weight of 120 pcf will be used for calculating the weight of the soil over a structure.

Bearing Capacity

An allowable bearing capacity of 2,000 psf will be used for the foundation design of the walls, considering that the foundation will be at least 24 inches below the existing grade, 24 inches in width, and be supported on a minimum of 24 inches of engineered fill. This value will be increased by one-third when considering loads of short duration, such as those imposed by wind or seismic forces. An allowable coefficient of friction value of 0.40 between the base of the footings and engineered fill can be used for sliding resistance using dead load forces.
Pile Foundations

CISS piles are the preferred foundation alternative for the widening of the bridge. Existing piles at the bridge consist of 16- and 36-inch-diameter CISS piles at the abutments and piers, respectively. For preliminary purposes, axial capacities for 16- and 36-inch CISS piles were calculated using Apile software (Ensoft, 2019a) for static and seismic conditions. The soil profile used in the pile analysis was based on existing nearby borings. Based on as-built plans for the previous widening of the bridge, a full scour depth of 12 feet was assumed at all piers for preliminary design purposes. Appendix B presents ultimate axial capacities for static and seismic conditions for abutments and piers.

4.4 Hydraulics Analysis

The hydraulics report highly depends on the geometry of the proposed widening and the added number of piers within the channel bed. Therefore, it was agreed to postpone the hydraulics analysis report until the structures team finalizes the proposed widening layout.

4.5 Structural Alternative Analysis

The existing Dominquez Channel structure was built circa 2001. The bridge structure is 204 feet long and 38 feet wide, carrying two railroad tracks and two 2-foot, 6-inch-wide walkways. Spans vary from 27.5 to 42 feet. The bridge is composed of 16 steel girders simply supported on 5 concrete bent caps. The bent caps are carried by five CISS piles. Due to the Dominquez Channel bed elevations, each pier column has a unique height, with Pier 4 the tallest.

The existing bridge was designed per ACTA (1999) Design Criteria for two seismic levels: operating level EQ (OLE) and contingency level EQ (CLE). Figure 4-4 shows the ARS curves for both levels of earthquakes. OLE is defined as an EQ event with a return period of 72 years in which the bridge responds without significant structural damage; that is, the extent of damage should be such that repairs can take place while continuing operation and maintaining safety. CLE is defined as an EQ event with a return period of 1,000 years in which the bridge responds with potential structural damage but survives without collapse. Table 4-4 summarizes ACTA seismic performance criteria.
Figure 4-4. ACTA Two-Level EQs Acceleration Response Spectrum

Table 4-4. ACTA Performance Criteria

<table>
<thead>
<tr>
<th>Seismic Level</th>
<th>Return Period (years)</th>
<th>Performance</th>
</tr>
</thead>
<tbody>
<tr>
<td>OLE</td>
<td>72</td>
<td>Elastic/Repairable Damage</td>
</tr>
<tr>
<td>CLE</td>
<td>1,000</td>
<td>Significant Damage/No Collapse</td>
</tr>
</tbody>
</table>

Figure 4-5 compares Level 1 response spectrum for the existing bridge with a 72-year return period and AREMA (2019) with a 100-year return period. The comparison shows that Level 1 ground motion acceleration has increased by approximately 90 percent within the bridge structural period range.
In this alternative, the bridge will be widened approximately 14 feet on the south side, extending beyond the railroad right-of-way by approximately 7 feet. The two existing tracks would remain in their present location, and the new third track would be added on the south side of the widened bridge. The final bridge configuration would be approximately 53 feet wide with two 2-foot, 6-inch-wide walkways.

This alternative matches the existing bridge configuration and structural elements sizes. A new pier cap with new 36-inch diameter CISS piles will be added to the south of the existing piers to support the new track. The new pile cap will be connected to existing pile cap using drill and bond or couplers. Similar to the existing bridge, the proposed widening is composed of six spans overcrossing the Dominguez Channel. Figure 4-6 provides a plan view depicting the existing and proposed bridge widening. Figure 4-7 and Figure 4-8 show the proposed bridge elevation and typical section, respectively.

The bridge superstructure is composed of steel plate girders simply supported on concrete piers. The steel plate girders are laterally connected with intermediate diaphragms. Intermediate diaphragms will be connected to existing steel plate girders using field welding and/or bolts. Appendix C includes a structural exhibit of the proposed widening. Proposed bridge loading and preliminary seismic analysis for the wider bridge configuration are discussed below.
Figure 4-6. Proposed Dominguez Channel Bridge Plan
Figure 4-7. Proposed Dominguez Channel Bridge Elevation
Preliminary structural seismic analysis was performed for the proposed widened bridge. The analysis focused on Level 1 ground motion in which the structure has to remain elastic per AREMA. The analysis was performed for both principal bridge directions: longitudinal and transverse. Dead loads and 25 percent of live loads were combined with the seismic loading. The 25 percent live load utilization was agreed upon with the rail team based on a recently performed utilization study for the bridge.

In the longitudinal direction, pier columns have a single curvature (i.e., maximum moment will occur at the bottom). Conversely, in the transverse direction, the pier columns act in a double curvature behavior (i.e., maximum moment will occur at the top and bottom). The widened bridge has a period of 0.56 and 0.50 second for transverse and longitudinal direction, respectively.

The pier CISS piles are 36 inches in diameter and filled with concrete. Longitudinal reinforcement is provided to transfer the moment from the CISS piles into the pile cap. As shown in Figure 4-9, there is insufficient embedment length for the steel pipe inside the pile cap; therefore, it was the top section considered as reinforced concrete, and steel pipe contribution is ignored. Based on the existing pile inground concrete embedment (see Figure 4-10), the maximum moment may
occur at a depth where concrete fill does not exist; therefore, only steel pile capacity for the inground moment is considered.

*Figure 4-9. Existing Pile Cap Connection Detail*
Figure 4-10. Existing Pile Inground Concrete Embedment
The piles were assumed to have equivalent length to points of fixity within the same pier. Points of fixity were based on Ensoft (2019b) Lpile models prepared by the geotechnical team, accounting for liquefaction and based on scour values reported by the hydraulics team (see Appendix B). Table 4-5 shows the estimated points of fixity for the piers during the preliminary seismic analysis.

Table 4-5. Piers Point of Fixity

<table>
<thead>
<tr>
<th>Support ID</th>
<th>Point of Fixity Depth/Pile Diameter</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>With Scour and Liquefaction</td>
</tr>
<tr>
<td>Pier 2</td>
<td>10</td>
</tr>
<tr>
<td>Pier 3</td>
<td>8</td>
</tr>
<tr>
<td>Pier 4</td>
<td>7</td>
</tr>
<tr>
<td>Pier 5</td>
<td>8</td>
</tr>
<tr>
<td>Pier 6</td>
<td>10</td>
</tr>
</tbody>
</table>

Preliminary results show that existing bridge piers will exceed capacity for the Level 1 EQ. For the transverse direction, the top piles capacity is exceeded by a demand to capacity ratio (DCR) of 1.26. However, although pile capacity is not reached for the longitudinal direction, the inground abutment piles capacity is reached.

AREMA only requires the Level 1 EQ to be checked for forces and the other two levels to be designed for ductile behavior. However, Level 2 EQ, as previously mentioned in Table 4-2, requires the damage to be accessible and repairable. Therefore, Level 2 EQ was evaluated and the abutment piles’ DCR increased to approximately 3, which means that inground hinging will occur. In addition, lateral spreading was evaluated in this analysis. Lateral spreading is expected to increase the DCR. Repair of the in-ground hinging for the abutment piles is not feasible. Therefore, additional widening alternatives are considered to meet AREMA Level 1, Level 2, and Level 3 performance requirements.

The presented preliminary results were based on simplified calculations without utilizing computer model. Further detailed analysis will be performed in the 30% Design to evaluate the south connected widening to the south side of the existing bridge. This will include three dimensional (3D) SAP2000 models to include bridge foundation, liquified soil, and lateral spreading.
4.5.1 Other Design Alternatives

Table 4-6 summarizes other alternatives considered for the Program and the challenges associated for each. These alternatives will be further evaluated in the next phase of the Program.

Table 4-6. Design Alternatives

<table>
<thead>
<tr>
<th>Alternative</th>
<th>Description</th>
<th>Challenge(s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Construct new widening to the south (connected to the existing bridge)</td>
<td>Widened bridge seismic displacements will be less than clearance to the existing south utility trestle</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Further detailed analysis will be performed in the 30% Design to evaluate this alternative</td>
</tr>
<tr>
<td>2</td>
<td>Remove existing bridge and construct new, wider bridge</td>
<td>Track utilization will be significantly interrupted during construction</td>
</tr>
<tr>
<td>3</td>
<td>Construct new widening to the north of existing bridge (connected or not connected to existing bridge)</td>
<td>Future potential POLA SCIG project will be affected</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Major track realignment is required</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Existing facility to the west end of the bridge will be affected</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Track crossovers if the widening is not connected</td>
</tr>
<tr>
<td>4</td>
<td>Remove south utility trestle and build new widening (not connected to the existing bridge)</td>
<td>Will involve third-party entity and will be very costly, challenging, and complex</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Track crossovers</td>
</tr>
<tr>
<td>5</td>
<td>Construct new widening to the south (connected to the existing bridge) and retrofit existing bridge; retrofit may include base isolation, modification to the abutment, and/or adding a new substructure to the north</td>
<td>Future potential POLA SCIG project will be affected</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Utility relocation</td>
</tr>
</tbody>
</table>

Notes: POLA = Port of Los Angeles; SCIG = Southern California International Gateway
4.6 Permitting

Table 4-7 provides a list of potential permits required for the bridge widening.

<table>
<thead>
<tr>
<th>Permit</th>
<th>Agency with Jurisdiction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Building Department Permit</td>
<td>COLA</td>
</tr>
<tr>
<td>Public Works Permit</td>
<td>COLA</td>
</tr>
<tr>
<td>Harbor Development Permit</td>
<td>POLA</td>
</tr>
<tr>
<td>USACE 208/408</td>
<td>USACE</td>
</tr>
<tr>
<td>Regional Water Quality 401</td>
<td>RWQCB</td>
</tr>
<tr>
<td>Coastal Development Permit</td>
<td>CCC</td>
</tr>
<tr>
<td>Flood Control Permit</td>
<td>LA County Flood Control District</td>
</tr>
</tbody>
</table>

Notes: CCC = California Coastal Commission; RWQCB = Regional Water Quality Control Board; USACE = United States Army Corps of Engineers

4.7 Record Documents

Table 4-8 summarizes the record documents reviewed for the structural analysis.

<table>
<thead>
<tr>
<th>Number</th>
<th>Record Document</th>
<th>Date</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Existing Bridge As-built Drawings (Full Set); see Appendix D</td>
<td>07/2000</td>
<td>ACTA</td>
</tr>
<tr>
<td>2</td>
<td>Existing Bridge Calculation and Independent Checker Calculations packages</td>
<td>04/24/2000</td>
<td>ACTA</td>
</tr>
<tr>
<td>3</td>
<td>Existing Bridge Final Foundation Report</td>
<td>04/25/2000</td>
<td>ACTA</td>
</tr>
</tbody>
</table>
5.0 SR-103 Crash Walls

The existing SR-103 (Terminal Island Freeway) structure was originally built circa 1947. The mainline structure is 1,544 feet long, consisting of two adjacent 19-span bridges and separated by a longitudinal deck joint. A segment of this viaduct spans over the western area of the Pier B Rail Yard, from Anaheim Way to I Street. Refer to Figure 3-1 for the SR-103 location.

The mainline 19-span bridge structure consists of reinforced concrete deck on composite continuous, alternate cantilever, and suspended riveted steel plate girders on riveted steel columns at Bents 13 and 14, and reinforced concrete columns at the remaining bents. The abutments are on steel piles and the columns are on untreated timber piles.

Within the Pier B Rail Yard, from Anaheim Way to I Street, the existing Bents 12, 13A and B, 14A and B, and 15 are aligned in parallel to I Street. Bents 13A and 13B are not aligned; they are offset by approximately 10 to 15 feet. The same condition applies to Bents 14A and 14B.

The Program proposes construction of new tracks, RD-1 through RD-6, with a horizontal clearance of approximately 10 to 12 feet to Bents 12, 13, and 14. As a result, new crash walls need to be constructed at these bents to limit damage by the redirection and deflection of railroad equipment to comply with AREMA (2019) Chapter 8 pier protection requirements. Figure 5-1 shows an overview of the Bridge Bents where crash walls will be constructed.

*Figure 5-1. SR-103 Freeway New Crash Walls Location*
5.1 Basis of Design

5.1.1 Applicable Codes, Standards, and Guidelines

The Program will be designed and constructed in accordance with the latest version of the following codes and standards:

- AREMA Manual for Railway Engineering (AREMA, 2019)
- Union Pacific Railroad/BNSF Railway Design Criteria (2016)

5.1.2 Materials

- Concrete: Minimum compressive strength at 28 days for CIP reinforced concrete crash walls and foundations = 4,000 PSI
- Reinforcing Steel: Will conform to the requirements of ASTM A706, Grade 60; all spiral reinforcement will conform to the requirements of ASTM A1064
- Structural Steel: Will conform to the requirements of ASTM A572, Grade 50

5.1.3 Loads and Load Combinations

- Dead Load (DC): Consists of crash wall and footing self-weight, and soil on top of footing
- Train Derailment Crash Load (CT): Per AREMA (2019) Chapter 8, 2.1.5.1c, the crash load per Method 1 in the report, Development of Crash Wall Design Loads from Theoretical Train Impact, is used for the design of the crash walls.
  \[
  F_{\text{crash}} = 600 \text{ kip (point load)} \text{ applied horizontally and normal to wall face at any point along the wall; the point load is applied at a height of 6 feet from top of rail}
  \]
- Tsunami Hazard Assessment Report (M&N, 2007) stated that for the bulk of the area within POLB, no water overtopping is expected; therefore, tsunami loads are not considered in the design
- Earthquake Load (EQ): The earthquake load is based on site-specific Caltrans ARS curve (1,000-year return period); the EQ earth pressure will have a triangular distribution with a maximum value of 18 pcf
- The load combinations are as follows:
  - 1.25 DC + 1.35 EV +1.50 EH + 1.75 LS
  - 1.25 DC + 1.35 EV + 1.50 EH + 1.40 WS


- 1.00 DC + 1.00 EV + 1.00 EH + 1.00 EQ
- 1.00 DC + 1.00 EV + 1.00 EH + 1.00 CT

where EV is vertical earth pressure, EH is horizontal earth pressure, LS is live load surcharge, and WS is wind load.

### 5.1.4 Sea Level Rise

The CRP prepared by POLB (2016) reported three scenarios for overtopping and inundation, with and without surge, for the 100-year return period. Table 5-1 summarizes the SLR for these scenarios at SR-103. The SLR is not considered in the design, as agreed upon with POLB.

Table 5-1. Sea Level Rise Impact at SR-103

<table>
<thead>
<tr>
<th>SLR Scenario</th>
<th>16-inch</th>
<th>36-inch</th>
<th>55-inch</th>
</tr>
</thead>
<tbody>
<tr>
<td>100-year Storm Surge Scenario Included</td>
<td>No</td>
<td>Yes</td>
<td>No</td>
</tr>
<tr>
<td>SR-103 Inundation</td>
<td>0–2 feet</td>
<td>2–4 feet</td>
<td>6–8 feet</td>
</tr>
</tbody>
</table>

* Indicates no potential water accumulation

### 5.1.5 Crash Walls to Tracks Clearance

The minimum required horizontal clearance from the exterior face of the crash wall to centerline of the nearest adjacent track is 10 feet at Bent 12, Bent 13, and Bent 14. At Bent 12, the new crash wall will be located to provide ±10-foot, 1-inch clearance.

Bents 13 and 14 each have a column where the minimum horizontal clearance from the column face to the adjacent track centerline is ±10 feet. As a result, there is insufficient space to build a crash wall in front of these columns for protection with a 10-foot minimum clearance. At both locations, the crash walls are laid out to have two segments intersected by the column and connected with a steel plate spanning between the two wall segments; refer to see Figure 5-2 for Bent 13 detail. For additional details for the crash walls, refer to Appendix C.
5.2 Existing Utilities and Right-of-Way Impact

5.2.1 Utilities and Potential Conflicts

There are two existing oil lines near Bent 14 that may conflict with the proposed shallow foundation depending on their depth below ground; these lines are considered a conflict if deep foundation is used. In addition, an abandoned 8-inch oil line is located at Bents 13 and 15. Appendix A provides the utility exhibit.

5.2.2 Right-of-Way Impact

No right-of-way impact is identified at this location; however, SR-103 is under Caltrans jurisdiction.

5.3 Geotechnical Considerations

5.3.1 Subsurface Conditions

Based on available boring information, subsurface materials at the site consist of up to 5 feet of artificial fill over natural deposits. Artificial fill materials are shown to generally consist of loose to medium dense silty sand and poorly graded gravel with silt. The artificial fill will be considered "uncertified" fill.
Natural deposits are shown to generally consist of low to high plasticity silts and clays with pockets of silty sand or poorly graded sand with silt. Silts and clays ranged from very soft to hard, and granular soils consisted of relative densities ranging from loose to dense within the upper 50 feet. A thin layer of elastic silt was encountered at depths between 16 and 17 feet bgs in one of the borings completed during the previous investigation. At depths greater than 50 feet bgs, subsurface soils generally consist of medium dense to dense sands to the maximum depths explored of 100 feet bgs. Appendix B provides historical borings and CPT logs.

5.3.2 Liquefaction

The Program site is located within an area designated as potentially liquefiable by the California Geological Survey (2020). Based on the site location being within a liquefaction zone and expected shallow groundwater depths, the potential for liquefaction exists at the site. Diaz Yourman Associates (DYA, 2011) estimated that the liquefaction-induced settlements would range from approximately 5 to 12 inches within the site. The crash walls will not be designed to mitigate liquefaction.

5.3.3 Compressible Soils

Soft clay layers were encountered within the upper 50 feet, and are considered to be moderately compressible when subjected to additional loads such as moderate to heavy foundation loads and/or additional fill soils. Static settlement will be re-evaluated during final design.

5.3.4 Corrosion

Based on corrosion test results provided by DYA (2011), the subsurface soils in the upper 5 feet at the site have low corrosion potential to buried concrete materials and are generally considered severely corrosive to buried ferrous metals.

5.3.5 Preliminary Geotechnical Recommendations

Seismic Design Spectra

Caltrans 1,000-year return period ARS curve was developed for Site Class E, assuming a weighted \( V_{s30} \) of 150 m/s using the ARS online tool (V2.3.09 [Caltrans, 2019]). This ARS curve is considered representative for this site and can be used in the preliminary design; see Appendix B.

Lateral Earth Pressure

A set of EFP values for three wall displacement conditions considering a level backfill are provided in Appendix B. The appropriate condition depends on the type of wall or shoring system selected and installation method. For example, a flexible sheet pile wall system may experience "Active"
conditions; a CIP diaphragm wall system might experience "At-Rest" conditions; and the resistance at the toe of the shoring might experience "Passive" conditions. Lateral earth pressures may be significantly higher for a sloped backfill condition; however, this condition is not anticipated for this site.

Surcharge loading from nearby active rail will be considered in the design of retaining structures. In addition to the above lateral pressures from retained earth, lateral pressures from other superimposed loads, such as those from adjacent structures or vehicles, will be added per AREMA (2019) Chapter 8, Section 5, and/or Caltrans (2011) Trenching and Shoring Manual Chapter 4. For surcharge loading onto wing walls or other retaining wall structures, loads will be calculated according to AREMA (2019) Chapter 8, Section 20.3.2. Preliminary surcharge pressure distribution using a live load surcharge for Cooper E80 is provided in Appendix B.

**Bearing Capacity**

An allowable bearing capacity of 2,000 psf will be used for the foundation design of the crash walls, considering that the foundation will be at least 24 inches below the existing grade, 24 inches in width, and supported on a minimum of 24 inches of engineered fill. This value will be increased by one-third when considering loads of short duration, such as those imposed by wind or seismic forces.

An allowable coefficient of friction value of 0.40 between the base of the footings and engineered fill can be used for sliding resistance using the dead load forces. Seismic settlements may cause damage and/or excessive settlement to structures of this type where soils are not remediated. Considering the relatively shallow depth of the artificial fill layer and the constraints mentioned above, it may be cost effective to remove and replace artificial fill with engineered fill. This will likely reduce the effects of seismic settlements on the crash walls as well. Alternatively, crash walls may be supported on a deep foundation system.

**5.4 Structural Alternative Analysis**

Per AREMA (2019) Chapter 8, Pier Protection Requirements, any column with clear distance to center line of the nearest track equal to or less than 25 feet will be protected by a 2-foot, 6-inch-thick crash wall. Based on this design criteria and the proposed tracks layout shown in Figure 5-3, four crash walls are required as follows:

- Bent 12 from the north side
- Bent 13 from the south and north sides
- Bent 14 from the south side
The height of all four crash walls will be 12 feet above top of rail as the clear distance to center line of the nearest track is less than 12 feet. The minimum length of the wall will be 12 feet, and the length of any wall will extend at least 1 foot beyond the outmost column parallel to the track. The crash walls will extend to at least 4 feet below the lowest surrounding grade. Appendix C provides structural exhibits of the proposed crash walls.

*Figure 5-3. SR-103 Freeway Existing Columns to Tracks Clearances*

---

**5.4.1 Structural Alternative 1: Spread Footing**

The existing bridge columns are located within 10 to 25 feet of the centerline of the adjacent new tracks at Bents 12, 13, and 14. The available space between the tracks and existing bridge footing for crash wall foundation construction is limited. This presents a challenge to designing a stand-alone crash wall system with a limited foundation footprint at each bent that can withstand large lateral crash loads isolated from the bridge footing.

Structural Alternative 1 proposes using spread footings with a width small enough to fit within the limited available space and designing a link beam or link slab to combine the spread footings into...
a larger system. This design increases the dead load of the wall system, which helps increase footing friction resistance. Additionally, the moment arm will increase, which helps the overturning stability. For extreme loading cases, a 33 percent increase for both soil bearing capacity and passive pressure per the preliminary geotechnical report is implemented. Per the calculation, seismic force is smaller than crash load; therefore, 600 kips lateral crash load governs the crash wall design. However, the wall is not designed for liquefied soil condition and associated effects such as settlement. Structural exhibits of the proposed crash walls for Alternative 1 are provided in Appendix C.

Pros and Cons

The following summarizes the pros and cons of Structural Alternative 1:

- **Pros:**
  - Crash walls spread footing width provides a minimum of 5 feet of clearance to the adjacent track to ease construction.
  - This alternative has relatively easier construction, with no deep excavation.
  - Crash walls are isolated from the existing bridge without interfering with the bridge footings and columns, resulting in no impact on the existing bridge.
  - This alternative has less interference with underground utilities.

- **Cons:**
  - Spread footings are connected by a beam/slab system, which requires irregular footing geometry that increases formwork for concrete construction.
  - This alternative has a heavily reinforced spread footing system due to irregular footing geometry.

5.4.2 Structural Alternative 2: Deep Foundation

This alternative proposes the use of stand-alone pile foundations for each wall. The crash walls are located beneath the existing bridge deck; therefore, overhead room for pile driving is limited. Cast-in-drill-hole (CIDH) piles are considered as a preferred option for deep foundation. Caltrans Standard Class 90 are used for this alternative, with nominal axial capacity of 180 kips and nominal lateral capacity of 32 kips. Structural exhibits of the proposed crash walls for Alternative 2 are provided in Appendix C.
Pros and Cons

The following summarizes the pros and cons of Structural Alternative 2:

- **Pros:**
  - Each wall has a stand-alone footing without connection to structure components with the existing crash wall; therefore, the footing would require lesser and easier formwork.
  - Walls are founded on a group pile cap; therefore, the depth of the wall embedded in the ground may be reduced.
  - Crash walls are isolated from the existing bridge without interfering with the bridge footings and columns, resulting in no impact on the existing bridge.

- **Cons:**
  - This alternative has difficult pile installation under the bridge superstructure due to available vertical clearance.
  - Stand-alone pile footing for each wall will need a wider pile cap, reducing the horizontal clear distance between edge of footing to the centerline of the adjacent track. This will make it difficult to construct if the track is in place before the crash walls are constructed.
  - This alternative has potential interference with existing utilities.

### 5.5 Permitting

Table 5-2 provides a list of potential permits required for the crash walls.

<table>
<thead>
<tr>
<th>Permit</th>
<th>Agency with Jurisdiction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Encroachment Permit</td>
<td>Caltrans</td>
</tr>
<tr>
<td>Harbor Development Permit</td>
<td>POLA</td>
</tr>
<tr>
<td>Coastal Development Permit</td>
<td>CCC</td>
</tr>
</tbody>
</table>
5.6 Record Documents

Table 5-3 summarizes the record documents reviewed for the structural analysis.

Table 5-3. Record Documents Reviewed

<table>
<thead>
<tr>
<th>Number</th>
<th>Record Document</th>
<th>Date</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Existing Bridge As-built Drawings (Full Set); Appendix D</td>
<td>09/19/1984</td>
<td>POLB</td>
</tr>
<tr>
<td>2</td>
<td>Existing Terminal Island Freeway Crash Walls; Appendix D</td>
<td>08/21/2003</td>
<td>POLB</td>
</tr>
<tr>
<td>3</td>
<td>Terminal Island Freeway Bridge Inspection Report</td>
<td>06/10/2008</td>
<td>POLB</td>
</tr>
</tbody>
</table>
6.0 Anaheim Street Overcrossing Tracks Support

New nine-rail tracks are proposed to be added underneath the Anaheim Overcrossing; refer to Figure 3-1 for the Anaheim Overcrossing location. Tracks cross the overcrossing at different elevations and clearances to the bridge columns. At three locations located at Bents 7 and 8, where tracks pass underneath the existing bridge, tracks cross over the existing bridge pile footing, and there is not enough vertical clearance to construct the ballast and rail assembly considering minimum required vertical clearance below the existing bridge. To resolve the issue, a shallow structure system (bridge-over) is proposed to span over the existing pile footing at these three locations. The bridge-over spans are approximately 30 feet long. Figure 6-1 shows one of the columns underneath the existing Anaheim Overcrossing.

*Figure 6-1. Anaheim Overcrossing Typical Column Overview*
6.1 Basis of Design

6.1.1 Applicable Codes, Standards, and Guidelines

The Program will be designed and constructed in accordance with the latest version of the following codes and standards:

- **AREMA Manual for Railway Engineering** (AREMA, 2019)

6.1.2 Materials

- Concrete: Minimum compressive strength at 28 days for CIP reinforced concrete foundations = 4,000 PSI
- Reinforcing Steel: Will conform to the requirements of ASTM A706, Grade 60; all spiral reinforcement will conform to the requirements of ASTM A1064
- Structural Steel: Will conform to the requirements of ASTM A992, Grade 50

6.1.3 Loads and Load Combinations

- Dead Load: Including self-weight and 200 pounds per linear foot track rails, inside guardrails and fastenings
- Live Load: Recommended live load for each track of main line structure is Cooper E80 loading
- Impact Load: Per AREMA (2019) Chapter 15, Section 1.3.5
- Longitudinal Force from Live Load: Per AREMA (2019) Chapter 15, Section 1.3.12
- Wind Load: On live load, per AREMA (2019) Chapter 15, Section 1.3.7
- Earthquake Load: The earthquake load is based on AREMA’s three earthquake levels.
- **Tsunami Hazard Assessment Report** (M&N, 2007) stated that for the bulk of the area within POLB, no water overtopping is expected; therefore, tsunami loads are not considered in the design
- The recommended load combinations will be based on AREMA (2019) as follows:
  - Chapter 8, Section 2.2.4, Table 8-2-4, SLD: Structural steel
  - Chapter 8, Section 2.2.4, Table 8-2-5, LFD: Concrete abutments
6.1.4 Sea Level Rise

The CRP prepared by POLB (2016) reported three scenarios for overtopping and inundation, with and without surge, for the 100-year return period. Table 6-1 summarizes the SLR for these scenarios at Anaheim Overcrossing. The SLR is not considered in the design, as agreed upon with POLB.

<table>
<thead>
<tr>
<th>100-year Storm Surge Scenario Included</th>
<th>16-inch</th>
<th>36-inch</th>
<th>55-inch</th>
</tr>
</thead>
<tbody>
<tr>
<td>No</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
</tr>
<tr>
<td>Anaheim Overcrossing Inundation</td>
<td>0–2 feet</td>
<td>2–4 feet</td>
<td>6–8 feet</td>
</tr>
</tbody>
</table>

* Indicates no potential water accumulation

6.2 Existing Utilities and Right-of-Way Impact

6.2.1 Utilities and Potential Conflicts

There are existing utilities at the location of these bridge-overs at Bents 7 and 8, including a 10-inch abounded gas line, 4-inch communication line, and 8-inch oil line. All these utilities are planned to be relocated. For the utility exhibit, refer to Appendix A.

6.2.2 Right-of-Way Impact

No right-of-way-impact is identified at this location; however, the Anaheim Overcrossing Bridge is under COLA jurisdiction.

6.3 Geotechnical Considerations

Preliminary geotechnical design recommendations are provided in the *Geotechnical Memorandum* prepared by HDR (2020); see Appendix B for details.

6.3.1 Subsurface Conditions

Subsurface conditions near the Program site were obtained from historical borings provided in the geotechnical reports prepared by Geofon (1994) and Leighton and Associates (1996). Based on available boring information, subsurface materials at the site consist of natural deposits. Natural deposits are shown to generally consist of loose to medium dense sand in the upper 7 to
13 feet bgs underlain by approximately 30 feet of clays and silts with occasional interlayers of sand. The silts and clays within this depth ranged in consistency between very soft to medium stiff. At depths greater than approximately 35 feet bgs, subsurface soils generally consist of medium dense to very dense sands to the maximum depths explored of 101.5 feet bgs.

### 6.3.2 Liquefaction

The Program site is located within an area designated as potentially liquefiable by the California Geological Survey (2020). Based on the site location being within a liquefaction zone and expected shallow groundwater depths, the potential for liquefaction exists at the site. Geofon (1994) estimated that the liquefaction-induced settlements would range from approximately 3 to 6 inches within the site. This evaluation will be updated prior to the final design once HDR’s geotechnical investigation is completed.

### 6.3.3 Compressible Soils

Soft clay layers were encountered within the upper 50 feet and are considered to be compressible when subjected to additional loads such as moderate to heavy foundation loads and/or additional fill soils. Static settlement will be evaluated during final design.

### 6.3.4 Corrosion

Based on the corrosion test results by Geofon (1994) and other nearby soil data, the subsurface soils at the site should be considered corrosive to buried concrete materials and ferrous metals. Protection measures will be identified in the Final PS&E. This includes using combination of coating and sacrificial thickness.

### 6.3.5 Seismic Design Spectra

ARS curves were developed for two boundary site classes (C/D and D/E) and a return period of 2,475 years using USGS’s (2020) unified hazard tool. Boundary of Site Class C/D with a $V_{s30}$ of 360 m/s was representative of a firm ground, resulting in higher spectral accelerations at lower spectral periods. Boundary of Site Class D/E with a $V_{s30}$ of 180 m/s was selected for the soft ground, resulting in higher spectral accelerations at higher periods. The preliminary ARS for the site was selected as the envelope of these two ARS curves and increased by 20 percent to account for softer than assumed soil profiles and the impact from the Wilmington Fault not considered in the USGS fault models.
6.3.6 Preliminary Geotechnical Recommendations

Pile Foundations

Axial capacities for a 15-inch-diameter cased CIDH pile and a 24-inch-diameter uncased CIDH pile were developed by the geotechnical team. The soil profile used in the pile analysis was based on existing nearby borings. Preliminary lateral capacities of piles based on deflections of 0.25, 0.5, and 1.0 inch for pinned head connections are presented in Table 6-2 for 24-inch uncased CIDH piles.

**Table 6-2. Summary of Lateral Pile Capacities for 24-inch Uncased CIDH Piles**

<table>
<thead>
<tr>
<th>Pile Capacity</th>
<th>Pile Head Deflection (Pin-top)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.25 inch</td>
</tr>
<tr>
<td>Lateral Capacity (kips)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>15</td>
</tr>
<tr>
<td>Flexural Depth (feet bgs)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>13.5</td>
</tr>
<tr>
<td>Maximum Moment (kip-inch)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>685</td>
</tr>
<tr>
<td>Depth to Max. Moment from Bottom of Pile Cap (feet)</td>
<td>6.5</td>
</tr>
</tbody>
</table>

6.4 Structural Alternative Analysis

Two bridge-over structure system alternatives were evaluated to span over the existing footings and to support the proposed tracks.

Alternative 1 uses prestressed concrete slab supported on seat-type concrete abutments. Based on the track alignment and profile, the minimum required vertical clearance for the tracks is 23 feet, 6 inches. This clearance will provide 20 inches of space above the existing bridge footing to construct the new prestressed concrete slab. Therefore, the maximum slab depth is limited to 15 inches. A prestressed concrete slab with 15-inch superstructure depth is deemed not feasible because it cannot support the track live load with a 30-foot span.

Alternative 2 uses a steel girders system supported on seat-type concrete abutments. Track live loads will be supported by steel cross beams spaced at 1.5-foot center-to-center that are supported on two longitudinal steel girders, as shown on Figure 6-2. The longitudinal girders will
be supported on two abutments seated on 24-inch CIDH piles. This alternative is considered feasible and can support the track live load with a 30-foot span.

*Figure 6-2. Alternative 2 Steel Girders System*

6.5 Permitting

A list of potential permits required for the bridge-over is provided in Table 6-3.

**Table 6-3. Potential Permit List for Anaheim Overcrossing Bridge-over**

<table>
<thead>
<tr>
<th>Permit</th>
<th>Agency with Jurisdiction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Public Works Permit</td>
<td>COLA</td>
</tr>
<tr>
<td>Harbor Development Permit</td>
<td>POLA</td>
</tr>
<tr>
<td>Coastal Development Permit</td>
<td>CCC</td>
</tr>
</tbody>
</table>
### 6.6 Record Documents

Table 6-4 summarizes the record documents reviewed for the structural analysis.

<table>
<thead>
<tr>
<th>Number</th>
<th>Record Document</th>
<th>Date</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Existing Bridge As-built Drawings; Appendix D</td>
<td>05/3/2020</td>
<td>POLB</td>
</tr>
<tr>
<td>2</td>
<td>Geotechnical Report Anaheim Street Grade Separation</td>
<td>11/21/1994</td>
<td>POLB</td>
</tr>
<tr>
<td>3</td>
<td>Anaheim ST OC Bridge Inspection Report</td>
<td>04/8/2016</td>
<td>POLB</td>
</tr>
</tbody>
</table>
7.0 Anaheim Street Overcrossing Retaining Wall

As part of the new added tracks at the Anaheim Street Overcrossing, a 26-foot service road is proposed to be added to the north of the new added tracks. The proposed service road alignment falls into the existing Anaheim Street Overcrossing Abutment 10 slope. Therefore, a retaining wall is proposed to retain the elevation difference of the proposed service road and the existing abutment slope. The retained height varies between 6 and 10 feet. Proposed utilities might interfere with the proposed wall layout. However, those utilities are deep enough to avoid the proposed wall foundation. The first proposed track centerline is located at 10 feet from the edge of the service road. Therefore, the proposed wall layout line is located at approximately 36 feet from the closest track centerline.

Figure 7-1. Pier B Street Proposed Retaining Wall
7.1 Basis of Design

7.1.1 Applicable Codes, Standards, and Guidelines

The Program will be designed and constructed in accordance with the latest version of the following codes and standards:

- Caltrans *Trenching and Shoring Manual* (Caltrans, 2011)
- Structural Concrete (American Concrete Institute [ACI] 318-14)

7.1.2 Materials

- Concrete: Minimum compressive strength at 28 days for CIP reinforced concrete wall, foundations = 4,000 PSI
- Reinforcing Steel: Will conform to the requirements of ASTM A706, Grade 60; all spiral reinforcement will conform to the requirements of ASTM A1064, Grade 36

7.1.3 Loads and Load Combinations

- The wall will be designed for the lateral earth pressures provided in Table 7-1.

<table>
<thead>
<tr>
<th>Condition</th>
<th>Equivalent Fluid Pressure (pcf) 2:1 Slope</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Level Backfill</td>
</tr>
<tr>
<td>Active</td>
<td>49</td>
</tr>
<tr>
<td>Passive</td>
<td>130 (to maximum 1,500 psf)</td>
</tr>
<tr>
<td>Seismic</td>
<td>45</td>
</tr>
</tbody>
</table>

- Soil unit weight is 120 pcf; therefore, \( k_a = \frac{49}{120} = 0.41 \). \( k_{AE} = \frac{45}{120} = 0.375 \)

- For seismic loading, a triangular pressure distribution of 45 pcf (EFP) will be used in addition to the static earth pressures and should be factored as appropriate per the geotechnical memorandum.

- *Tsunami Hazard Assessment Report* (M&N, 2007) stated that for the bulk of the area within POLB, no water overtopping is expected; therefore, tsunami loads are not considered in the design.
The recommended load combinations, based on AASHTO (2017) Chapter 3, Section 3.3.2, Table 3.4.1-1, Load Combinations and Load Factors, will be implemented for the wall design.

### 7.1.4 Sea Level Rise

The CRP prepared by POLB (2016) reported three scenarios for overtopping and inundation, with and without surge, for the 100-year return period. Table 7-2 summarizes the SLR for each scenario at the Anaheim Street Overcrossing Retaining Wall. The SLR is not considered in the design, as agreed upon with POLB.

<table>
<thead>
<tr>
<th>SLR Scenario</th>
<th>16-inch</th>
<th>36-inch</th>
<th>55-inch</th>
</tr>
</thead>
<tbody>
<tr>
<td>100-year Storm Surge Scenario Included</td>
<td>No</td>
<td>Yes</td>
<td>No</td>
</tr>
<tr>
<td>Anaheim Street Overcrossing Retaining Wall Inundation</td>
<td>a 0–2 feet</td>
<td>b 2–4 feet</td>
<td>6–8 feet</td>
</tr>
</tbody>
</table>

*a Indicates no potential water accumulation  
*b Indicates the structure will be below water surface without a direct hydraulic flow path

### 7.2 Existing Utilities and Right-of-Way Impact

#### 7.2.1 Utilities and Potential Conflicts

Few proposed utilities conflict with the proposed wall layout. However, these utilities are deeper than the proposed wall foundation. For the utility exhibit, refer to Appendix A.

#### 7.2.2 Right-of-Way Impact

No right-of-way-impact is identified at this location; however, the Anaheim Overcrossing Bridge is under COLA jurisdiction.

### 7.3 Geotechnical Considerations

#### 7.3.1 Subsurface Conditions

Subsurface conditions in the area of the proposed wall generally consist of a fill embankment supporting the existing Anaheim Street abutment approach, ranging in elevation from...
approximately EI +30 feet, NAVD88, down to the natural ground surface around +8 feet. The embankment fill materials were considered to consist of Caltrans engineered fill standard materials consisting of a sand with a 34-degree friction angle and 120 pcf unit weight. Beneath the fill embankment, native on-site soils begin at around elevation +8 feet. An upper loose to medium-dense sand layer is generally encountered down to approximately EI 0. Beneath this sand layer, a zone of loose sands and very soft to stiff clays and silts were encountered to about EI -40. Beneath EI -40, generally dense sands were encountered. Groundwater in the area of the wall was encountered at approximately EI +3 feet. Groundwater levels are likely to fluctuate and be tidally influenced, and a preliminary design groundwater EI +5 feet is recommended; see Appendix B for further information.

7.3.2 Liquefaction

Based on preliminary analyses performed on two borings and one cone penetration test, it is estimated that the potential for liquefaction does exist, from approximately EI +5 to -40. Estimated seismic settlement at these three locations from an AREMA Level II seismic event are in the range of approximately 6 to 7 inches. Liquefaction mitigation using ground improvement is recommended.

7.3.3 Seismic Design Spectra

Based on the wall’s proposed design, both AREMA (2019) and County of Los Angeles (2020) design criteria were considered in selection of ground motions for the analyses. The typical AREMA (2019) and Southern California Regional Rail Authority (SCRRA, 2014) seismic design requirements utilize one-half of AREMA Seismic Level II for slope stability analyses, and the same ground motion is similarly used for development of earth pressures. Seismic lateral earth pressure is provided in Appendix B.

7.3.4 Bearing Capacity

The proposed wall footing is supported on soft and liquefiable soils. Due to these and slope stability considerations, it is recommended to support the wall on improved ground. An allowable bearing capacity of 4,000 psf may be assumed for shallow foundation design, considering that the foundation will be supported on improved ground and built to Caltrans standard dimensions. The above-mentioned allowable bearing capacity may be increased by one-third when considering loads of short duration, such as those imposed by wind or seismic forces. An allowable coefficient of friction value of 0.50 between the base of the footings and ground improved zone can be used for sliding resistance using the dead load forces.
7.4 Structural Alternative Analysis

The wall serves the purpose of retaining an elevation difference of 6 to 10 feet between the proposed service road and the existing Anaheim Street Overcrossing Bridge Abutment slope. The proposed service road is located at the lower elevation; therefore, the proposed wall retains mainly lateral earth pressure from the existing abutment slope. The nearest track is located approximately 36 feet away. The proposed wall uses a Caltrans Type 1 Retaining Wall. The wall design height varies from 6 to 10 feet. The retaining wall foundation is required to be supported by a minimum of 2 feet of engineered fill. Figure 7-2 shows a typical section of the proposed alternative. For the structural exhibit, refer to Appendix C. The wall extends to the west side until the elevation difference is approximately 2 feet, which can be supported by k-rail. On the east side, the wall extends until the elevation difference can be sloped at a 2:1 slope. To avoid the liquefaction potential at this site and to provide enhanced stability for the existing abutment slope, the proposed retaining wall is supported on ground improvement as recommended by the geotechnical memorandum.

*Figure 7-2. Anaheim Street Overcrossing Proposed Retaining Wall Section*
7.5 Permitting

A list of potential permits required for the Pier B Street retaining wall is provided in Table 7-3.

Table 7-3. Permits List for Bridge Widening

<table>
<thead>
<tr>
<th>Permit</th>
<th>Agency with Jurisdiction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Public Works Permit</td>
<td>COLA</td>
</tr>
<tr>
<td>Harbor Development Permit</td>
<td>POLA</td>
</tr>
<tr>
<td>Coastal Development Permit</td>
<td>CCC</td>
</tr>
</tbody>
</table>

7.6 Record Documents

Table 7-4 summarizes the record documents reviewed for the structural analysis.

Table 7-4. Record Documents Reviewed

<table>
<thead>
<tr>
<th>Number</th>
<th>Record Document</th>
<th>Date</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Existing Bridge As-built Drawings, Appendix D</td>
<td>05/3/2020</td>
<td>POLB</td>
</tr>
</tbody>
</table>
8.0 Pier B Street Retaining Wall

As part of the Pier B Rail Yard improvements, Pier B Street is proposed to be realigned and widened from a two-lane facility to a four-lane street. The street will generally be widened towards the north with a 4-foot sidewalk. To accommodate the widened street, a retaining wall is required at the north side, adjacent to the new Service Road of Rail Yard 1; refer to Figure 3-1 for the retaining wall location. The roadway layout dictates that the retaining wall begin near Lineage Logistics and end near Andeavor Logistics, for an approximate length of 1,600 feet. The site generally slopes down from the existing Pier B Street to the existing rail yard. The maximum retained height proposed to separate Pier B Street and the rail yard service road is approximately 5 feet for the exposed face of the wall. Figure 8-1 shows the proposed layout line of the retaining wall. There are extensive utilities (both overhead and underground) along Pier B and in the vicinity, including oil lines, drainage, electrical, and others.

From the north side of the wall, a 20-foot-wide service road is located at a 5-foot clearance from the layout line. The first proposed track centerline is located at 10 feet from the edge of the service road. Therefore, the proposed wall layout line is located at approximately 35 feet from the closest track centerline. From the south side, the wall layout line is approximately 32 feet away from the centerline of realigned Pier B Street.

Figure 8-1. Pier B Street Proposed Retaining Wall
8.1 Basis of Design

8.1.1 Applicable Codes, Standards, and Guidelines

The Program will be designed and constructed in accordance with the latest version of the following codes and standards:

- Caltrans *Trenching and Shoring Manual* (Caltrans, 2011)
- Structural Concrete (ACI 318-14)

8.1.2 Materials

- Concrete: Minimum compressive strength at 28 days for CIP reinforced concrete wall, foundations = 4,000 PSI
- Reinforcing Steel: Will conform to the requirements of ASTM A706, Grade 60; all spiral reinforcement will conform to the requirements of ASTM A1064, Grade 36

8.1.3 Loads and Load Combinations

- The wall will be designed for the lateral earth pressures provided in Table 8-1.

<table>
<thead>
<tr>
<th>Condition</th>
<th>Equivalent Fluid Pressure (pcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Level Backfill</strong></td>
<td></td>
</tr>
<tr>
<td>Active</td>
<td>40</td>
</tr>
<tr>
<td>At-Rest</td>
<td>58</td>
</tr>
<tr>
<td>Passive</td>
<td>300 (to maximum 3,000 psf)</td>
</tr>
</tbody>
</table>

- For surcharge loading onto retaining wall structures, loads will be calculated according to AREMA (2019) Chapter 8, Section 20.3.2. For preliminary purposes, a uniform vertical surcharge pressure of 240 psf (2 feet of soil), and a uniform rectangular pressure distribution of 250 psf can be used for vehicular traffic. A surcharge pressure distribution using a live load surcharge due to Cooper E80 is presented in the *Geotechnical Memorandum*; see Appendix B.
- Tsunami Hazard Assessment Report (M&N, 2007) stated that for the bulk of the area within POLB, no water overtopping is expected; therefore, tsunami loads are not considered in the design.

- For seismic loading, a triangular pressure distribution of 52 pcf (EFP) will be used in addition to the static earth pressures and should be factored as appropriate. This seismic earth pressure will be assumed to act with a similar load distribution as static pressures, and is applicable for both cantilever and braced conditions. Forces resulting from wall inertia effects are expected to be relatively minor for non-gravity walls and may be ignored in estimating the seismic lateral earth pressure.

- The recommended load combinations, based on AASHTO (2017) Chapter 3, Section 3.3.2, Table 3.4.1-1, Load Combinations and Load Factors, will be implemented for the wall design.

### 8.1.4 Sea Level Rise

The CRP prepared by POLB (2016) reported three scenarios for overtopping and inundation, with and without surge, for the 100-year return period. Table 8-2 summarizes the SLR for each scenario at the Pier B Street Retaining Walls. The SLR is not considered in the design, as agreed upon with POLB.

<table>
<thead>
<tr>
<th>SLR Scenario</th>
<th>16-inch</th>
<th>36-inch</th>
<th>55-inch</th>
</tr>
</thead>
<tbody>
<tr>
<td>100-year Storm Surge Scenario Included</td>
<td>No</td>
<td>Yes</td>
<td>No</td>
</tr>
<tr>
<td>Pier B Street Retaining Wall Inundation</td>
<td>0–2 feet</td>
<td>2–4 feet</td>
<td>6–8 feet</td>
</tr>
</tbody>
</table>

*a Indicates no potential water accumulation

*b Indicates the structure will be below water surface without a direct hydraulic flow path

### 8.2 Existing Utilities and Right-of-Way Impact

#### 8.2.1 Utilities and Potential Conflicts

Multiple existing underground utilities run parallel to the proposed retaining wall alignment. For the utility exhibit, refer to Appendix A.
8.2.2 Right-of-Way Impact

No right-of-way-impact is identified at this location; however, the proposed retaining wall would require right-of-way coordination with the City of Long Beach (COLB) and POLB.

8.3 Geotechnical Considerations

8.3.1 Subsurface Conditions

Based on available boring information, subsurface materials at the site generally consist of natural deposits with some locations indicating minor fill soils in the upper several feet. Fill soils generally consisted of clayey sand with gravel. Natural deposits are shown to generally consist of loose to medium dense sand in the upper 10 feet bgs underlain by approximately 35 feet of clays and silts with occasional interlayers of sand. The silts and clays within this depth ranged in consistency between very soft and medium stiff. The boring performed by DYA indicated that at depths greater than approximately 45 feet bgs, subsurface soils generally consist of medium dense to very dense sands to the maximum depths explored of approximately 100 feet bgs. However, intermittent layers of clay, silt, and sand were encountered during a recent boring by HDR; see Appendix B for further information.

8.3.2 Liquefaction

The Program site is located within an area designated as potentially liquefiable by the California Geological Survey (2020). Based on the site location being within a liquefaction zone and expected shallow groundwater depths, the potential for liquefaction exists at the site. Preliminary calculations estimate that soils in the upper 50 feet bgs may be susceptible to liquefaction, and liquefaction-induced settlements would be up to approximately 7 inches within the site. This evaluation will be updated during future design stages.

8.3.3 Corrosion

Based on experience with other portions of the Program site and existing site data, highly corrosive soil should be anticipated. Corrosion measures to protect steel reinforcement will be implemented.

8.3.4 Seismic Design Spectra

ARS curves were developed for two boundary site classes (C/D and D/E) and a return period of 2,475 years using USGS’s (2020) unified hazard tool. Boundary of Site Class C/D, with a $V_{s30}$ of 360 m/s, was representative of a firm ground, resulting in higher spectral accelerations at lower spectral periods. Boundary of Site Class D/E with a $V_{s30}$ of 180 m/s was selected for the soft
ground, resulting in higher spectral accelerations at higher periods. The preliminary ARS for the site was selected as the envelope of these two ARS curves increased by 20 percent to account for softer than assumed soil profiles and the impact from the Wilmington Fault not being considered in the USGS fault models. Appendix B provides the preliminary ARS curve for the return period of 2,475 years.

8.3.5 Bearing Capacity

The bearing capacity of the retaining wall defined in the geotechnical report is 1,500 psf considering the foundation will be at least 24 inches below the existing grade, a minimum of 7.5 feet in width, and supported on a minimum of 24 inches of engineered fill. The ability to increase the bearing capacity is feasible by additional over-excavation and construction of a geogrid-and-gravel-reinforced “raft” foundation beneath the wall; however, this has not been considered in the current design. The allowable bearing capacity will be increased by one-third when considering loads of short duration, such as those imposed by wind or seismic forces. For the calculation of base friction between the foundation and soil, the allowable coefficient of friction is recommended to be 0.4.

8.4 Structural Alternative Analysis

The wall serves the purpose of retaining an elevation difference of 5 feet between the proposed tracks and the realigned Pier B Street. Pier B Street is located at the higher elevation; therefore, the proposed wall retains mainly lateral earth pressure and the surcharge due to vehicular loading. From the other side, 20-foot maintenance access is located between the wall and the nearest track. The nearest track is located approximately 35 feet away. Two structural alternatives were evaluated.

Alternative 1 uses a Mechanical Stabilized Embankment (MSE) wall system. Figure 8-2 shows a typical section of the proposed alternative.

The advantage of using an MSE wall for this location includes the flexibility it provides during a seismic event, which increases the chances that the wall may stay serviceable after a moderate earthquake. However, MSE wall design includes soil reinforcement, which conflicts with multiple existing underground utilities that run parallel to the proposed wall. Therefore, this alternative is not feasible.

Alternative 2 uses a Caltrans Type 1 Retaining Wall. The wall design height varies from 6 to 8 feet. The retaining wall foundation is required to be supported by a minimum of 2 feet of engineered fill. Figure 8-3 shows a typical section of the proposed alternative. For the structural exhibit, refer to Appendix C.
8.5 Permitting

A list of potential permits required for the Pier B Street retaining wall is provided in Table 8-3.

<table>
<thead>
<tr>
<th>Permit</th>
<th>Agency with Jurisdiction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Building Permit</td>
<td>COLB</td>
</tr>
<tr>
<td>Harbor Development Permit</td>
<td>POLB</td>
</tr>
<tr>
<td>Division Land Use Permit</td>
<td>POLB</td>
</tr>
</tbody>
</table>

8.6 Record Documents

No record documents were reviewed for the structural analysis.
9.0 LA-04 Pump Station

The Program proposes to relocate the existing West Long Beach Pump Station (LA-04 Pump Station) located mid-block of 9th Street between Harbor and Caspian Avenues. The existing facility is used to convey flows from the LA-04 drainage area and has a 54-inch outfall that discharges into Channel No. 2. The facility is owned and operated/maintained by the LA County Department of Public Works. Figure 9-1 shows the existing pump station location and the proposed location of the new pump station.

Figure 9-1. Pump Station Proposed Relocation

The proposed pump station is located on the west side of the 54-inch force main to minimize the length of the connection to the force main with access from Pier B Street. The site offers ample space on either side to serve as the pump station yard and laydown area during the construction phase. The proposed plan and section views for the pump station are shown in Figure 9-2 and Figure 9-3, respectively. The proposed building consists of a 59- by 30-foot wet well, upon which a 41- by 50-foot above-ground structure encloses the pumps and related infrastructure.
Figure 9-3. Pump Station Proposed Section
9.1 Basis of Design

9.1.1 Applicable Codes, Standards, Guidelines, and Structures and Architectural Considerations

The Program will be designed and constructed in accordance with the latest version of the following codes and standards:

- California Building Code (CBC, 2019)
- Building Code Requirements and Specification for Masonry Structures (ACI 530 2013)
- Requirements for Structural Concrete (ACI 318-14)
- Specification for Structural Steel Buildings (American Institute of Steel Construction [AISC] 360-16)
- Seismic Provisions for Structural Steel Buildings (AISC 341-16)
- Minimum Design Loads and Associated Criteria for Buildings and Other Structures with Supplement No. 1 (ASCE 7-16)
- Building Code for Masonry Structures (The Masonry Society [TMS] 402-16)
- Specification for Masonry Structures (TMS 602-16)
- Structural Welding Code – Seismic Supplement (AWS D1.8/D1.8M -2016)
- Structural Welding Code – Sheet Steel (AWS D1.3/D1.3M-08)
- Structural Welding Code – Reinforcing Steel Including Metal Inserts and Connections in Reinforced Concrete Construction (D1.4/D1.4M- 2017)
- California Electrical Code (CEC)
- California Energy Code (CEN)
- California Fire Code (CFC)
- California Green Building (CGB) Standard Code
- California Mechanical Code (CMC)
- California Occupational Safety and Health Administration (CalOSHA) General Industry Orders
- COLB Municipal Code
National Fire Protection Association (NFPA)

9.1.2 Materials

- Concrete: Minimum compressive strength at 28 days for CIP reinforced concrete walls, slabs, foundations and CISS (Grade A252) pile in-fill = 5,000 PSI
- Reinforcing Steel: Will conform to the requirements of ASTM A706 and A615, Grade 60; all spiral reinforcement will conform to the requirements of ASTM A1064
- Masonry: Hollow concrete blocks will conform to the requirements of ASTM C90, Grade N
- Structural Steel: Will conform to the requirements of ASTM A36
- Sheet Pile: Will conform to the requirements of ASTM A572, Grade 60
- Steel Metal Deck: Will conform to the requirements of ASTM A653

9.1.3 Loads and Load Combinations

- Dead Loads: This includes vertical loads due to the weight of permanent structural and nonstructural components of a building or structure. Table 9-1 shows the minimum unit weight for all materials.

<table>
<thead>
<tr>
<th>Material</th>
<th>Minimum Dead Load (pcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Backfill</td>
<td>Per Geotechnical Memorandum; Appendix B</td>
</tr>
<tr>
<td>Reinforced Concrete</td>
<td>150</td>
</tr>
<tr>
<td>Steel</td>
<td>490</td>
</tr>
<tr>
<td>Masonry</td>
<td>102</td>
</tr>
<tr>
<td>Water</td>
<td>63</td>
</tr>
<tr>
<td>Bitumen</td>
<td>150a</td>
</tr>
</tbody>
</table>

*a Roof membrane of the building in the pump station will be of the modified bitumen type; conservatively, the thickness of this layer corresponds to 4 inches

- Equipment Load: A conservative pump and motor weight of 17,100 pounds (lbs) is considered for this design stage and provided by the hydraulics team. Equipment loads will have the same load factor as dead loads.
- Live Loads: This includes loads produced by the use and occupancy of the building or structure. They include the weight of all movable loads, including personnel, tools, miscellaneous equipment, parts of dismantled equipment, stored material, floor area loads, equipment handling loads, laydown loads, truck wheel loads, and similar items.

Live loads and reduction of live loads will be computed according to applicable code. The floor area live load will be omitted from areas occupied by equipment under which no access is provided and whose weight has been included in dead load. Live load will be included under equipment where access is provided.

Table 9-2 provides the minimum live loads to be used in the design, unless established by analysis of specific area loads.

**Table 9-2. Minimum Design Live Loads**

<table>
<thead>
<tr>
<th>Component</th>
<th>Uniform (psf)</th>
<th>Concentrated (lbs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Corridors: First floor</td>
<td>100</td>
<td>N/A</td>
</tr>
<tr>
<td>Roofs: Assembly Areas</td>
<td>100</td>
<td>N/A</td>
</tr>
<tr>
<td>Scuttles, Skylight Ribs, and Accessible Ceilings</td>
<td>N/A</td>
<td>200</td>
</tr>
<tr>
<td>Stairs and Exits</td>
<td>100</td>
<td>300</td>
</tr>
</tbody>
</table>

Notes: N/A = not applicable

The indicated concentrated loads will be assumed to be uniformly distributed over an area of 2.5 by 2.5 feet, and will be located to produce the maximum load effects in the structural members.

- Equipment Reactions: The design will consider the reactions of the pumps and the motor. Equipment reactions will have the same load factor as live loads. The design values will be finalized in the final design.
- Traffic Loads: Installations accessible to truck loading will be designed to withstand truck loads. Truck loads will have the same load factor as live loads.
- Truck HS-20: The truck has the characteristics shown in Figure 9-4. Design will consider a total length of 28 feet.
- Impact load: This will be 33 percent of live load.
- Crane: The design must consider the load of a Link-Belt Model HTC8640HL Telescopic Boom Truck Crane.
- Rail Load: This will be per AREMA for rail tracks surcharge load.
- Wind Loads: Wind conditions on the site will be determined per CBC:
  - Risk Category: III
  - Basic Wind Speed (V): 115 MPH
  - Wind Directionality Factor (Kd): 0.9
  - Exposure Category: D
  - Topographic Factor (Kzt): 1
  - Enclosure Classification: “Building Partially Enclosed”
  - Internal Pressure Coefficient: 0.55
  - Velocity Pressure Exposure Coefficient (Kz): 1
  - Velocity Pressure (qz): 30.47 psf
- Rain Loads (R): The roof will be designed to sustain the load of rainwater, per CBC, that will accumulate on it if the primary drainage system for that portion is blocked plus the uniform load caused by water that rises above the inlet or the secondary drainage system at its design flow.
  - Additional Depth of Water: 16 inches
  - 100-year, 1-hour Rainfall: 3 inches
o R: 98.8 psf

- Flood Loads: Due to flooding at the site location, a hydrostatic pressure consistent with a water level at EI +19.0 feet, NAVD88, will be considered.
- *Tsunami Hazard Assessment Report* (M&N, 2007) stated that for the bulk of the area within POLB, no water overtopping is expected; therefore, tsunami loads are not considered in the design.
- Water Pressure: Due to variations in water level at the site location, water pressure will be considered.
  o The maximum water elevation will be considered at ground surface (EI +13.0 feet, NAVD88). The minimum water elevation will be considered at EI +2.0 feet, NAVD88.
- Operation Water Pressure: Due to variations in operation water level at pump station, only two cases are considered: full water depth and empty case.
- Resonance: The potential structure resonance caused by the rotating water pumps will be verified, and the need for skid grouting to mitigate it will be identified.
- Buoyancy: Due to variations in water level at the site location, the buoyant force acting on the wet well structure will be considered. The maximum and minimum water elevations are identified in the *Geotechnical Memorandum* in Appendix B.
- Lateral Earth Pressures: The lateral load value is shown in Table 9-3; see Appendix B.

**Table 9-3. Equivalent Fluid Pressure Values**

<table>
<thead>
<tr>
<th>Condition</th>
<th>Equivalent Fluid Pressure (pcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td><strong>Above Groundwater</strong></td>
</tr>
<tr>
<td>Active</td>
<td>40</td>
</tr>
<tr>
<td>At-Rest</td>
<td>58</td>
</tr>
<tr>
<td>Passive</td>
<td>300 (to maximum 3,000 psf)</td>
</tr>
</tbody>
</table>
• Earthquake Load: Based on the *Geotechnical Memorandum* included in Appendix B, the seismic design spectra is shown in Figure 9-5 for 5 percent damping.

*Figure 9-5. Design Spectra*

![Design Spectra Graph]

### 9.1.4 Sea Level Rise

The CRP prepared by POLB (2016) reported three scenarios for overtopping and inundation, with and without surge, for the 100-year return period. Table 9-4 summarizes the SLR for each scenario at the LA-04 Pump Station. The SLR is not considered in the design, as agreed upon with POLB.

**Table 9-4. Sea Level Rise Impact on LA-04 Pump Station**

<table>
<thead>
<tr>
<th></th>
<th>SLR Scenario</th>
<th>16-inch scenario</th>
<th>36-inch scenario</th>
<th>55-inch scenario</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>100-year Storm Surge Scenario Included</strong></td>
<td>No</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
</tr>
<tr>
<td><strong>LA-04 Pump Station Inundation</strong></td>
<td>-a</td>
<td>0–2 feet</td>
<td>-a</td>
<td>2–4 feet</td>
</tr>
</tbody>
</table>

*a Indicates no potential water accumulation*
9.2 Existing Utilities and Right-of-Way Impact

9.2.1 Utilities and Potential Conflicts

The proposed pump station consists of two main structures: the underground wet well and above-ground building. Based on the proposed layout and the utilities in proximity with the wet well, two potential conflicts have been identified. The first is a 4-inch communication line owned by Verizon that will need to be relocated during construction. The second is a 6-inch wet gas line owned by LOMITA. This line does not interfere directly with the proposed wet well footprint; however, it will need to be relocated due to its proximity to the expected construction footprint. Appendix A provides an exhibit showing utilities that fall within the proposed pump station layout.

9.2.2 Right-of-Way Impact

The LA-04 Pump Station will be relocated to a location where rail tracks currently exist within the POLB right-of-way but will be owned and operated by LA County Department of Public Works. Coordination of this encroachment will need to be addressed.

9.3 Geotechnical Considerations

9.3.1 Subsurface Conditions

Based on available information, subsurface materials at the site consist of a thin upper layer of fill underlain by natural deposits. Approximately 5 feet of fill consisting of silty sand with gravel was noted at the surface. Beneath this layer, a sand layer grading from dense to loose extends to a depth of approximately 14 feet bgs. A very soft silt/clay layer is encountered next, extending to a depth of approximately 29 feet bgs. Beneath this layer, alternating zones of sands, silts, and clays were encountered to the maximum depth explored of approximately 81.5 feet bgs. Relative density/consistency of the soils generally increased with depth and became dense/hard or stronger at a depth of approximately 70 feet (approximately El -61 feet, NAVD88). Appendix B provides borings logs, along with generalized soil profile with preliminary design parameters.

9.3.2 Liquefaction

The Program site is located within an area designated as potentially liquefiable by the California Geological Survey (2020). Based on the site location being within a liquefaction zone and expected shallow groundwater depths, as well as preliminary analyses results, the site is expected to experience liquefaction during the design earthquake. This evaluation will be updated prior to the final design once HDR’s geotechnical investigation is completed.
9.3.3 Corrosion

No historical corrosion data are available for review; however, based on experience, fine-grained soils are generally more likely to be corrosive, and the subsurface soils at the site may be corrosive to buried concrete materials and ferrous metals.

9.3.4 Compressible Soils

Soft clay layers were encountered within the upper 40 feet and are considered to be compressible when subjected to additional loads such as moderate to heavy foundation loads and/or additional fill soils. Static settlement will be re-evaluated during final design.

9.3.5 Seismic Design Spectra

ARS curves were developed for two boundary site classes (C/D and D/E) and a return period of 2,475 years using USGS’s (2020) unified hazard tool. Boundary of Site Class C/D with a $V_{s30}$ of 360 m/s was representative of a firm ground, resulting in higher spectral accelerations at lower spectral periods. Boundary of Site Class D/E with a $V_{s30}$ of 180 m/s was selected for the soft ground, resulting in higher spectral accelerations at higher periods. The preliminary ARS for the site was selected as the envelope of these two ARS curves increased by 20 percent to account for softer than assumed soil profiles and the impact from the Wilmington Fault not being considered in the USGS fault models. Appendix B shows the preliminary ARS curve for the return period of 2,475 years.

9.3.6 Geotechnical Recommendations

Foundation/Construction Type

Based on available plans for construction of the pump station and that active dewatering is not allowed by POLB due to potential for contaminated groundwater migration, two general approaches are considered feasible. These approaches are to use 1) a pile-supported foundation system with a sheet pile shoring system, or 2) a ground improvement program consisting of deep soil mixing or a similar ground improvement.

- Pile-Supported Foundation with Sheet Pile Construction: This method utilizes piles extending beneath the bottom of the proposed pump station to support the structure’s vertical and lateral loading, including potential uplift, and to mitigate the effects of downdrag.
- Ground Improvement with Deep Soil Mixing: This method is the preferred method of construction and includes improving the ground within the construction area using deep soil mixing or a similar method. Deep soil mixing utilizes large diameter augers equipped
with cement injection nozzles that mix cement with soil as they penetrate into the soil strata. The result will be a strengthened, non-liquefiable column of cement-soil mix that is also hydraulically impermeable (in relative terms).

Lateral Earth Pressure

Appendix B provides a set of EFPS for the preliminary design of the proposed wall, considering a level backfill. The appropriate condition depends on the type of wall and installation method.

Bearing Capacity

Structures that are considered “inhabited” should not be founded on spread footings without ground improvement due to liquefaction and static settlement concerns. Where ground improvement is used, the resulting allowable bearing capacities will generally be high. Details on spread footing design can be provided as the design progresses and construction method is selected.

Pile Foundations

A deep foundation alternative is discussed in the Geotechnical Memorandum (Appendix B). Driven piles are preferred by POLB due to environmental issues pertinent to drilled piles. Based on evaluation of the subsurface information available, driven piles will likely tip at approximately E1 -75 feet, NAVD88. Driven steel piles should consider the corrosive environment present and may require sacrificial steel thickness or other corrosion protection. Precast, prestressed concrete driven piles have also been used successfully in many projects in the area. Driven, 14-inch, square, precast concrete piles would have ultimate axial compression capacities in the range of 400 kips and uplift capacities in the range of 60 kips.

Pile lateral capacities are likely to be minor in comparison to the lateral pressures/capacities of the pump station walls. However, if the pile foundation alternative is selected, these capacities can be developed during future design stages. Capacities will depend on pile size, type, and location (beneath pump station foundation versus at-grade for office building support).

9.4 Architectural Considerations

The pump station building will be placed above the wet well structure. The wet well is considered a confined space, and access stairs to the wet well will follow CalOSHA. The pump station building will follow CBC with a construction Type IIIB, non-sprinklered. The occupancy type will be F-2. The building will have a minimum of one exit. The site and building will be designed for complete accessibility to meet the minimum requirements of CBC Chapter 11 (and other chapters) and the Americans with Disabilities Act (ADA). This includes providing accessible toilet, parking space, and path of travel to the building entrance(s).
The building will be of masonry construction, and the masonry will be a smooth-face type, integrally colored. The roof will have masonry parapets and will be a tapered insulation roof construction. The roof membrane will be modified bitumen type.

The doors and any louvers will be of steel construction. The entry will have a steel canopy and steel accents. All roof drainage will be exterior type drainage with downspouts and overflows. The roof will have removable skylights and additional roof solatube openings for additional daylighting. The removable skylights will be functional as they will be required to be removed when the pump cans get pulled in the future for maintenance or replacement. The building will have an interior roof ladder for maintenance access. All lighting will be light-emitting diode (LED). The interior ceiling will have an exposed finish but be painted. All exposed steel will be galvanized and painted.

### 9.5 Structural Alternative Analysis

The proposed pump station is composed of two main structures: the underground wet well and the above-ground building. Based on the proposed layout by the drainage team, the wet well needs to be accessible; therefore, stairs are extended to the bottom of the wet well.

Based on geotechnical recommendations and that active dewatering is not allowed by POLB due to potential for contaminated groundwater migration, two general approaches are considered feasible. These approaches are to use 1) a pile-supported foundation system with a sheet pile shoring system, or 2) a ground improvement program consisting of deep soil mixing or a similar ground improvement. Both alternatives are considered along with the construction sequencing required for each alternative. See Appendix C for structural exhibits.

The challenges were identified for the wet well design and construction as follows:

- The ground water table is located approximately at the ground surface, which means that the wet well is submerged in water. This will cause huge uplift forces that will tend to push the wet well up.
- The site has high potential for liquefaction during an extreme event. This is a critical aspect as it may cause the pump station to be out of service after seismic activity.
- Active dewatering is not allowed by POLB due to potential for contaminated groundwater migration. This is mostly a construction challenge; however, it also affects the design approach.

#### 9.5.1 Construction Alternative 1: Pile-supported Foundation System

In this alternative, piled foundation is used to address the first two challenges stated above: uplift and potential liquefaction. Special detailing is needed between the proposed piles and wet well to prevent uplift. This will be further addressed in the next design phase. Additionally, piled
foundation will be specified with enough embedment to mitigate liquefaction potential. Figure 9-6 shows a cross-section for the proposed alternative.

Walls will be designed to support lateral soil pressure in addition to water pressure. Additionally, any surcharge loading from the surrounding roadway and/or trains will be accounted for. Steel stair beams will be supported on 8-foot-long cantilever slabs that will be supported on the wet well walls. The above-ground building roof will be designed for specified loading per CBC.

*Figure 9-6. Pump Station Proposed Construction Alternative 1*

**Construction Sequence**

To address the limitation regarding active dewatering, it is proposed to use sequential construction using a sheet pile wall. Table 9-5 summarizes this construction sequence, which includes driving sheet pile walls around the wet well perimeter followed by excavating the area surrounded by the sheet pile wall. It is expected the ground water table will be high in the wet well excavated footprint. The next step includes pile driving to tip at approximately EL -75 feet, NAVD88. During any of the previous construction stages, dewatering will not be allowed as this would require active dewatering. Tremie concrete layer pouring to mitigate the water transfer up through the wet well footprint occurs next. At this stage, water will be removed from the wet well location as the water passage is mitigated with the sheet pile wall and tremie concrete layer. The traditional construction sequence would follow for the wet well floor, walls, stairs, and ceiling. Finally, the sheet pile wall may be removed.
## Table 9-5. Alternative 1 Construction Sequence

<table>
<thead>
<tr>
<th>Construction Stage</th>
<th>Stage Description</th>
<th>Schematic</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Insert sheet pile wall around the wet well perimeter</td>
<td><img src="image1" alt="Schematic" /></td>
</tr>
<tr>
<td>2</td>
<td>Excavate soil to reach tremie concrete level</td>
<td><img src="image2" alt="Schematic" /></td>
</tr>
</tbody>
</table>
### Table 9-5. Alternative 1 Construction Sequence

<table>
<thead>
<tr>
<th>Construction Stage</th>
<th>Stage Description</th>
<th>Schematic</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>Drive piles to tip at El -75 feet, NAVD88 (estimated)</td>
<td><img src="image1" alt="Schematic" /></td>
</tr>
<tr>
<td>4</td>
<td>Pour tremie concrete layer</td>
<td><img src="image2" alt="Schematic" /></td>
</tr>
</tbody>
</table>
### Table 9-5. Alternative 1 Construction Sequence

<table>
<thead>
<tr>
<th>Construction Stage</th>
<th>Stage Description</th>
<th>Schematic</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>Remove water within the wet well footprint</td>
<td><img src="image1" alt="Schematic" /></td>
</tr>
<tr>
<td>6</td>
<td>Build wet well floor and walls</td>
<td><img src="image2" alt="Schematic" /></td>
</tr>
</tbody>
</table>
### Table 9-5. Alternative 1 Construction Sequence

<table>
<thead>
<tr>
<th>Construction Stage</th>
<th>Stage Description</th>
<th>Schematic</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>Build wet well ceiling, stairs, and above-ground building</td>
<td>![Schematic Image]</td>
</tr>
</tbody>
</table>
Pros and Cons

The following summarizes the pros and cons of Construction Alternative 1.

- **Pros:**
  - Fairly traditional construction methodology/equipment
  - Relatively more rapid construction time
  - Smaller footprint required for construction versus ground improvement

- **Cons:**
  - Increased potential for water migration during construction
  - Potential for needing tie-back/strut support and limitations associated with support
    (limited available height above water to install struts/tiebacks and constrained work area if using struts)
  - Corrosive environment to foundation piles
  - Potential construction issues related to installation of piles within a pool and underwater non-level slab pour adjacent to shoring, which may prove difficult

### 9.5.2 Construction Alternative 2: Deep Soil Mixing

This alternative proposes ground improvement with deep soil mixing. It proposes improving the ground within the construction area using deep soil mixing, which utilizes large diameter augers equipped with cement injection nozzles that mix cement with soil as they penetrate into the soil strata. The result will be a strengthened, non-liquefiable column of cement-soil mix that is also relatively hydraulically impermeable. The soil improvement is proposed to stop at the wet well bottom elevation. However, for the area around the wet well footprint, the soil mixing will extend to ground surface. Figure 9-7 shows a cross-section for the proposed alternative.

This alternative is believed to resolve the dewatering challenge as it will effectively mitigate the water seepage through the ground improved column. Additionally, liquefaction potential is addressed using this alternative. All wet well structural components, including floor, walls, and ceiling, will be designed using the same approach indicated in Alternative 1.
Construction Sequence

The proposed construction sequence includes constructing deep soil mix in a “bathtub” shape, with some areas extending to a bottom of approximately EI -70 feet, NAVD88 (estimated). This is followed by excavating the center of the “bathtub” shore or bracing internally if additional lateral support is needed (water infiltration into excavated area is expected to be minimal). Finally, traditional construction methods will be used from the bottom up with no need for deep foundations. The construction sequence for this alternative is summarized in Table 9-6.
<table>
<thead>
<tr>
<th>Construction Stage</th>
<th>Stage Description</th>
<th>Schematic</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-a</td>
<td>Develop deep soil ground improvement to bottom elevation of approximately El -70 feet, NAVD88 (estimated)</td>
<td><img src="image" alt="Schematic" /></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1-b</td>
<td>Develop deep soil ground improvement at the wet well footprint with top elevation equal to wet well bottom elevation</td>
<td><img src="image" alt="Schematic" /></td>
</tr>
</tbody>
</table>
### Table 9-6. Alternative 2 Construction Sequence

<table>
<thead>
<tr>
<th>Construction Stage</th>
<th>Stage Description</th>
<th>Schematic</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>Excavate soil to reach wet well bottom elevation</td>
<td><img src="image1" alt="Schematic" /></td>
</tr>
<tr>
<td>3</td>
<td>Remove water within wet well footprint</td>
<td><img src="image2" alt="Schematic" /></td>
</tr>
</tbody>
</table>
## Table 9-6. Alternative 2 Construction Sequence

<table>
<thead>
<tr>
<th>Construction Stage</th>
<th>Stage Description</th>
<th>Schematic</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>Build wet well floor and walls</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Build wet well ceiling, stairs, and above-ground building</td>
<td></td>
</tr>
</tbody>
</table>
Pros and Cons

The following summarizes the pros and cons of this method.

- **Pros:**
  - Resolves issues pertinent to foundation support, liquefaction, dewatering, and shoring in one step
  - Involves a relatively straightforward method of construction without complexities related to the pile-supported alternative

- **Cons:**
  - Development of some semi-cementitious spoils during construction
  - Possible longer construction timeframes
  - Requirement for a larger footprint area for cement batching and mixing equipment

### 9.6 Permitting

Table 9-7 provides a list of potential permits required for the pump station.

<table>
<thead>
<tr>
<th>Permit</th>
<th>Agency with Jurisdiction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Building Permit</td>
<td>COLB</td>
</tr>
<tr>
<td>Division Land Use Permit</td>
<td>POLB</td>
</tr>
<tr>
<td>Harbor Development Permit</td>
<td>POLB</td>
</tr>
<tr>
<td>Coastal Development Permit</td>
<td>CCC</td>
</tr>
<tr>
<td>Review and Approval</td>
<td>LA County Flood Control District</td>
</tr>
</tbody>
</table>
9.7 Record Documents

Table 9-8 summarizes the record documents reviewed for the structural analysis.

<table>
<thead>
<tr>
<th>Number</th>
<th>Record Document</th>
<th>Date</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Existing West Long Beach (LA-04) Pump Station</td>
<td>10/1969</td>
<td>LA County Department of Public Works</td>
</tr>
<tr>
<td></td>
<td>As-built drawings; Appendix D</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Ocean Boulevard Storm Water Pump Station</td>
<td>11/21/1997</td>
<td>Moffatt &amp; Nichol Library</td>
</tr>
</tbody>
</table>
10.0  Sewer Lift Station

A new sewer lift station is proposed at the corner of Anaheim and 9th Streets in COLB. Refer to Figure 3-1 for the proposed Sewer Lift Station location. The sewer lift station is composed of two vaults: the lift station vault has an inner dimension of 12 by 24 feet, and the valve vault has an inner dimension of 5 by 24 feet. Figure 10-1 shows the proposed sewer lift station layout. The sewer lift station is proposed to have deeper invert elevation than the valve vault. Figure 10-2 shows a cross-section in the vaults.

Figure 10-1. Proposed Sewer Lift Station Layout

Figure 10-2. Proposed Sewer Lift Station Cross-section
## 10.1 Basis of Design

### 10.1.1 Applicable Codes, Standards, and Guidelines

The Program will be designed and constructed in accordance with the latest version of the following codes and standards:

- **AREMA Manual for Railway Engineering** (AREMA, 2019)
- **CBC** (2019)
- **COLB Municipal Code**
- **Requirements for Structural Concrete** (ACI 318-14)
- **Minimum Design Loads and Associated Criteria for Buildings and Other Structures with Supplement No. 1** (ASCE 7-16)

### 10.1.2 Materials

- Concrete: Minimum compressive strength at 28 days for CIP reinforced concrete foundations = 4,000 PSI
- Reinforcing Steel: Will conform to the requirements of ASTM A706, Grade 60; all spiral reinforcement will conform to the requirements of ASTM A1064

### 10.1.3 Loads and Load Combinations

- Dead Loads: Vertical loads due to the weight of permanent structural and nonstructural components of a building or structure. Table 10-1 shows the minimum unit weight for all materials.

#### Table 10-1. Minimum Design Dead Loads

<table>
<thead>
<tr>
<th>Material</th>
<th>Minimum Dead Load (pcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Backfill</td>
<td>Per Geotechnical Memorandum; Appendix B</td>
</tr>
<tr>
<td>Reinforced Concrete</td>
<td>150</td>
</tr>
<tr>
<td>Water</td>
<td>63</td>
</tr>
</tbody>
</table>

- Live Loads: Loads produced by pedestrian traffic and exposure of the top slab of the sewer lift station. They include the weight of all movable loads, including personnel and equipment handling loads. Live loads will be computed according to applicable code.
Table 10-2 shows the minimum live loads to be used in the design, unless established by analysis of specific area loads.

Table 10-2. Minimum Design Live Loads

<table>
<thead>
<tr>
<th>Component</th>
<th>Uniform (psf)</th>
<th>Concentrated (pounds)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top slab</td>
<td>100</td>
<td>Not Applicable</td>
</tr>
</tbody>
</table>

- Flood Loads: Due to flooding at the site location, a hydrostatic pressure consistent with a water level at EI +19.0 feet, NAVD88, will be considered; see Section 10.1.4.

- Tsunami Hazard Assessment Report (M&N, 2007) stated that for the bulk of the area within POLB, no water overtopping is expected; therefore, tsunami loads are not considered in the design.

- Water Pressure: The maximum water elevation will be considered at ground surface (EI +13.0 feet, NAVD88). The minimum water elevation will be considered at EI +2.0 feet, NAVD88.

- Operation Water Pressure: Two cases are considered: full water depth and empty case.

- Buoyancy: The buoyant force acting on the wet well structure will be considered. The Geotechnical Memorandum (Appendix B) identifies the maximum and minimum water elevations.

- Lateral Earth Pressures: The lateral load value is shown in Table 10-3; see Appendix B.

Table 10-3. Equivalent Fluid Pressure Values

<table>
<thead>
<tr>
<th>Condition</th>
<th>Equivalent Fluid Pressure (pcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Above Groundwater</td>
</tr>
<tr>
<td>Active</td>
<td>45</td>
</tr>
<tr>
<td>At-rest</td>
<td>65</td>
</tr>
<tr>
<td>Passive</td>
<td>300 (to maximum 3,000 psf)</td>
</tr>
<tr>
<td>Seismic</td>
<td>50</td>
</tr>
</tbody>
</table>

- Earthquake: Based on triangular pressure distribution with the EFP identified in the Geotechnical Memorandum included in Appendix B. This load will be used in addition to the static earth pressures and should be factored in. This seismic earth pressure will be
assumed to act with a similar load distribution as static pressures, and is applicable for both cantilever and braced conditions.

- Rail Load: AREMA for rail tracks surcharge load
- The recommended load combinations, based on CBC Chapter 16, Load Combinations Using Strength Design or Load and Resistance Factor Design, will be implemented for the wet well and above-ground building design.

10.1.4 Sea Level Rise

The CRP prepared by POLB (2016) reported three scenarios for overtopping and inundation with and without surge for a 100-year return period. Table 10-4 summarizes the SLR for each scenario at the sewer lift station. The SLR is not considered in the design, as agreed upon with POLB.

Table 10-4. Sea Level Rise Impact on Sewer Lift Station

<table>
<thead>
<tr>
<th>SLR Scenario</th>
<th>16-inch</th>
<th>36-inch</th>
<th>55-inch</th>
</tr>
</thead>
<tbody>
<tr>
<td>100-year Storm Surge Scenario Included</td>
<td>No</td>
<td>Yes</td>
<td>No</td>
</tr>
<tr>
<td>Sewer Lift Station Inundation</td>
<td>-a</td>
<td>0–2 feet</td>
<td>-a</td>
</tr>
</tbody>
</table>

* Indicates no potential water accumulation

10.2 Existing Utilities and Right-of-Way Impact

10.2.1 Utilities and Potential Conflicts

The proposed sewer lift station consists of two underground vaults. Based on the proposed layout, no known utilities are at the sewer lift station location; refer to Appendix A.

10.2.2 Right-of-Way Impact

No right-of-way-impact is identified at this location; however, the proposed sewer lift station falls within COLB’s jurisdiction.
10.3 Geotechnical Considerations

10.3.1 Subsurface Conditions

Borings indicated pavement materials and/or fill in the upper 3 to 4 feet. The borings indicated somewhat differing soil profiles, with borings generally indicating soft to stiff fine-grained soils with occasional sands until depths ranging from approximately 30 to 70 feet bgs, where sand was encountered. Because the three borings are only separated by about 260 feet, the subsurface soil profile should be considered highly spatially variable. Appendix B provides a generalized profile with preliminary soil design parameters and subsurface cross section.

10.3.2 Liquefaction

The Program site is located within an area designated as potentially liquefiable by the California Geological Survey (2020). Based on the site location being within a liquefaction zone and expected shallow groundwater depths, the potential for liquefaction exists at the site. The Leighton and Associates (1996) report indicates that approximately 2 to 10 inches of seismically-induced settlement can be expected at the site. This evaluation will be updated during future design stages.

10.3.3 Corrosion

Soils in the Program area are generally considered corrosive. Additionally, the Leighton and Associates (1996) report indicates that subsurface soils are considered corrosive to buried concrete and ferrous metals. Therefore, the subsurface soils at the site should be considered corrosive to buried concrete materials and ferrous metals. Corrosion measures to protect steel reinforcement will be implemented.

10.3.4 Compressible Soils

Soft clay layers were encountered within the upper 30 to 70 feet, and are considered to be compressible when subjected to additional loads such as moderate to heavy foundation loads and/or additional fill soils. Static settlement will be re-evaluated during the final design. Based on the proposed construction at this site, large vertical loads are not expected and, therefore, consolidation is not expected to be a design issue.

10.3.5 Seismic Design Spectra

ARS curves were developed for two boundary site classes (C/D and D/E) and a return period of 2,475 years using USGS’s (2020) unified hazard tool. Boundary of Site Class C/D with a $V_{s30}$ of 360 m/s was representative of a firm ground, resulting in higher spectral accelerations at lower
spectral periods. Boundary of Site Class D/E with a $V_{530}$ of 180 m/s was selected for the soft ground, resulting in higher spectral accelerations at higher periods. The preliminary ARS for the site was selected as the envelope of these two ARS curves increased by 20 percent to account for softer than assumed soil profiles and the impact from the Wilmington Fault not considered in the USGS fault models. Appendix B shows the preliminary ARS curve for the return period of 2,475 years.

### 10.3.6 Geotechnical Recommendations

#### Foundation/Construction Type

The proposed structure will be designed to resist uplift loading and will require mitigation of settlement due to liquefaction. However, the adjacent piping or other shallow improvements connecting to the sewer lift station may not have these requirements; therefore, consideration should be given to the potential for differential settlements at connections to unprotected improvements that may include flexible connections or other solutions.

#### Lateral Earth Pressure

Appendix B provides a set of EFPs for the preliminary design of the proposed wall considering a level backfill. The appropriate condition depends on the type of wall and installation method.

#### Bearing Capacity

Structures that are considered “inhabited” should not be founded on spread footings without ground improvement due to liquefaction and static settlement concerns. Where ground improvement is used, the resulting allowable bearing capacities will be generally high. Details on spread footing design can be provided as the design progresses and construction method is selected.

#### Pile Foundations

Driven piles are preferred by POLB due to environmental issues pertinent to drilled piles. Based on the subsurface information available, driven piles will likely tip at approximately EL -70 feet, NAVD88. Driven steel piles should consider the corrosive environment present and may require sacrificial steel thickness or other corrosion protection. Precast, prestressed concrete driven piles have also been used successfully in many projects in the area. Driven 14-inch square precast concrete piles would have ultimate axial compression capacities in the range of 400 kips and uplift capacities in the range of 60 kips.
10.4 Structural Alternative Analysis

The proposed sewer lift station is composed of two underground vaults. Active dewatering is not allowed by POLB due to potential for contaminated ground water migration. Two alternatives are considered for foundation design to address liquefaction and no active dewatering during construction. Alternative 1 uses pile-supported foundation system. This alternative is considered not feasible due to the need for an extensive shoring system to excavate and construct the underground vaults to prevent active dewatering. Alternative 2 uses a ground improvement foundation system; refer to Appendix C for the structural exhibits. This alternative’s challenges are as follows:

- The ground water table is located approximately 5 feet bgs, which means that most of the vault is submerged under water. This will cause uplift forces that need to be considered in the design.
- Ground improvement will mitigate settlement due to liquefaction. However, the adjacent piping connecting to the sewer lift station will not have ground improvements; therefore, consideration should be given to the potential differential settlements at piping connections. The piping connections may need to have flexible connections design.
- The vaults’ design needs to accommodate for the not permitted active dewatering during construction.

Due to the size of the vaults and for ease of construction, a precast concrete structure type is proposed. Vault walls will be designed to support lateral soil and hydrostatic pressures as well as surcharge load due to vehicular and rail live loads. Deep soil mixing is proposed as a ground improvement method. Deep soil mix utilizes large diameter augers equipped with cement injection nozzles. These augers mix cement with soil as they penetrate into the soil strata. The result will be a strengthened, non-liquefiable column of cement-soil mix that is also relatively hydraulically impermeable. Figure 10-3 shows a cross-section for the proposed alternative.
10.5 Permitting

Table 10-5 provides a list of potential agencies that may require permits for the sewer lift station.

Table 10-5. Permits List for Sewer Lift Station

<table>
<thead>
<tr>
<th>Permit</th>
<th>Agency with Jurisdiction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Building Permit</td>
<td>COLB</td>
</tr>
<tr>
<td>Division Land Use Permit</td>
<td>POLB</td>
</tr>
<tr>
<td>Harbor Development Permit</td>
<td>POLB</td>
</tr>
<tr>
<td>Sewer Connection Permit</td>
<td>LA County Sanitation District</td>
</tr>
</tbody>
</table>

10.6 Record Documents

No record documents were reviewed for the structural analysis.
11.0 SR-710 Retaining Wall

The ongoing Gerald Desmond Bridge Replacement (GDB) project has constructed two walls along southbound SR-710, named as C4 and Q Walls. There is a gap between the two walls where no wall is constructed. The SR-710 ramp bridge footing is located on the soil slope to the north of the existing C4 Wall. Refer to Figure 3-1 for the retaining walls location. Figure 11-1 shows the C4 and Q Walls constructed as part of GDB project.

Figure 11-1. Gerald Desmond Bridge Retaining Wall Q and C4 (Existing Condition)

The proposed Program includes adding new tracks along Pico Avenue. The proposed layout has a total of eight tracks along southbound SR-710. Figure 11-2 shows an overview for the area and the proposed wall layout line. To create the required horizontal clearance for the proposed track close to southbound SR-710, the soil slope needs to be retained. Many underground utilities are located underneath the existing C4 Wall. This is covered in more detail in the next sections.

A site visit was conducted on December 12, 2019, to identify the above-ground constraints and potential conflicts. Refer to the Pier B Site Investigation Report 1-2-3 (HDR, 2019) for further details.
11.1 Basis of Design

11.1.1 Applicable Codes, Standards, Guidelines

The Program will be designed and constructed in accordance with the latest version of the following codes and standards:

- AREMA Manual for Railway Engineering (AREMA, 2019)
- Caltrans SDC V 2.0 (Caltrans, 2019)
- Caltrans Highway Design Manual – Structures
- Caltrans Design Information Bulletin (DIB) 82-06
- Caltrans STPs
- ACI 318-14

11.1.2 Materials

- Concrete: Minimum compressive strength at 28 days for CIP reinforced concrete foundations = 4,000 PSI
- Reinforcing Steel: Will conform to the requirements of ASTM A706, Grade 60; all spiral reinforcement will conform to the requirements of ASTM A1064
- Sheet Pile: Will conform to the requirements of ASTM A572, Grade 60
11.1.3 Loads and Load Combinations

- The retaining walls will be designed for the lateral earth pressures in Table 11-1.

**Table 11-1. Equivalent Fluid Pressure Values**

<table>
<thead>
<tr>
<th>Condition</th>
<th>Equivalent Fluid Pressure (pcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Level Backfill</td>
</tr>
<tr>
<td>Active</td>
<td>37</td>
</tr>
<tr>
<td>At-rest</td>
<td>56</td>
</tr>
<tr>
<td>Passive</td>
<td>360 (to maximum 3,600 psf)</td>
</tr>
<tr>
<td>Seismic</td>
<td>33</td>
</tr>
</tbody>
</table>

- Surcharge loading from vehicular traffic and active rail will be considered in the design of retaining walls. The specified loads will be per AREMA (2019) Chapter 8, Section 5, and/or Caltrans (2011) *Trenching and Shoring Manual*, Section 6. Surcharge loading will be determined according to AREMA Chapter 8, Section 20.3.2. Uniform rectangular pressure distribution of 250 psf is used for vehicular traffic.

- Seismic ARS curve: Caltrans Site class E (1,000-year return period)

- *Tsunami Hazard Assessment Report* (M&N, 2007) stated that for the bulk of the area within POLB, no water overtopping is expected; therefore, tsunami loads are not considered in the design.

- Load combinations according to AASHTO Table 3.4.1-1, Load Combinations and Load Factors, will be used in the retaining wall design.

11.1.4 Sea Level Rise

The CRP prepared by POLB (2016) reported three scenarios for overtopping and inundation with and without surge for a 100-year return period. Table 11-2 summarizes the SLR for each scenario at the SR-710 walls. The SLR is not considered in the design, as agreed upon with POLB.
Table 11-2. Sea Level Rise Impact on SR-710 Retaining Walls

<table>
<thead>
<tr>
<th>100-year Storm Surge Scenario Included</th>
<th>SLR Scenario</th>
<th>16-inch</th>
<th>36-inch</th>
<th>55-inch</th>
</tr>
</thead>
<tbody>
<tr>
<td>No</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
<td>No</td>
</tr>
<tr>
<td>SR-710 retaining walls Inundation (from rail side)</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>0–2 feet</td>
</tr>
</tbody>
</table>

* Indicates no potential water accumulation

11.1.5 Retaining Walls to Tracks Clearance

Minimum horizontal clearance of 10 feet is required from the exterior face of the proposed retaining wall to the centerline of nearest adjacent track.

11.2 Existing Utilities and Right-of-Way Impact

11.2.1 Utilities and Potential Conflicts

The initial utility investigation identified 21 utilities that will be impacted by the construction of the new retaining wall. However, the GDB project has removed and/or abandoned some of the existing utilities, reducing the number of potential utility conflicts to 12. There are seven active utilities that are perpendicular to the C4 retaining wall layout line as shown in Figure 11-3. Refer to Appendix A for the utilities exhibit.

*Figure 11-3. Underground Utilities at Proposed SR-710 Retaining Walls*
11.2.2 Right-of-Way Impact

The proposed retaining walls are in the Caltrans right-of-way and under their jurisdiction. Some portions of the proposed walls are outside the Caltrans right-of-way. Therefore, additional right-of-way will need to be transferred from POLB to Caltrans.

11.3 Geotechnical Considerations

The Geotechnical Memorandum prepared for the SR-710 walls location presents the preliminary geotechnical design recommendations for the proposed wall; see Appendix B for details.

11.3.1 Subsurface Conditions

Based on available boring information, subsurface materials at the site consist of up to approximately 25 feet of artificial fill at the SR-710 embankments over natural deposits. Artificial fill materials are shown to generally consist of medium dense silty sand. Natural deposits are shown to generally consist of fine sands, silts, and clays. Underlying the fill, layers of soft to medium stiff clays and silts were encountered to a depth of approximately 70 feet bgs, corresponding to approximate EL -30 feet, NAVD88. Layers of medium dense to very dense sands (silty sand and poorly graded sand) and very stiff to hard silts were encountered to a depth of approximately 130 feet bgs, corresponding to approximately EL -90 feet, NAVD88. Below a depth of approximately 130 feet bgs, medium stiff to stiff silts were encountered to the maximum depth explored of approximately 170 feet bgs, corresponding to approximately EL -130 feet, NAVD88. Appendix B includes a preliminary soil subsurface profile.

11.3.2 Liquefaction

The Program site is located within an area designated as potentially liquefiable by the California Geological Survey (2020). However, based on preliminary analysis provided in the Caltrans (2020) memorandum prepared for the Harbor Scenic Drive Bridge, located immediately to the north of the proposed SR-710 walls, soils within 70 feet bgs (measured from the top of embankment) are non-liquefiable. This evaluation will be updated prior to the final design once HDR’s geotechnical investigation is completed.

11.3.3 Corrosion

Corrosion data was not provided in the documents available for review. Based on the geotechnical team’s experience, fine-grained soils are generally more likely to be corrosive. Since subsurface native soils consist of silts and clays from depths ranging between 25 and 70 feet bgs, it is possible the site soils may be corrosive to buried concrete and ferrous metals. Corrosion protection measures, including sacrificial steel thickness and cathodic protection, will be considered in final design.
11.3.4 Preliminary Geotechnical Recommendations

Seismic Design Spectra

ARS curves were developed for two boundary site classes (C/D and D/E) and a return period of 975 years using USGS’s (2020) unified hazard tool. Boundary of Site Class C/D with a $V_{s30}$ of 360 m/s was representative of a firm ground, resulting in higher spectral accelerations at lower spectral periods. Boundary of Site Class D/E with a $V_{s30}$ of 180 m/s was selected for the soft ground, resulting in higher spectral accelerations at higher periods. The preliminary ARS for the site was selected as the envelope of these two ARS curves increased by 20 percent to account for softer than assumed soil profiles and the impact from the Wilmington Fault not considered in the USGS fault models. Appendix B includes the preliminary ARS curve for the return period of 975 years.

Lateral Earth Pressure

Appendix B provides EFP values for three wall displacement conditions considering a level backfill. The appropriate condition depends on the type of wall or shoring system selected, and on the installation method. For example, a flexible sheet pile wall system might experience "Active" conditions; a CIP diaphragm wall system might experience "At-rest" conditions; and the resistance at the toe of the shoring might experience "Passive" conditions.

Surcharge loading from nearby vehicular traffic and active rail will be considered in the design of retaining structures. In addition to the above lateral pressures from retained earth, lateral pressures from other superimposed loads, such as those from adjacent structures or vehicles, will be added per AREMA (2019) Chapter 8, Section 5, and/or Caltrans (2011) Trenching and Shoring Manual, Section 6. For surcharge loading onto retaining wall structures, loads will be calculated according to AREMA (2019) Chapter 8, Section 20.3.2. For preliminary purposes, a uniform rectangular pressure distribution of 250 psf can be used for vehicular traffic. Appendix B presents a surcharge pressure distribution using a live load surcharge due to Cooper E80.

For seismic loading, a triangular pressure distribution with the value presented in Appendix B will be used in addition to the static earth pressures and should be factored as appropriate. This seismic earth pressure will be assumed to act with a similar load distribution as static pressures, and is applicable for both cantilever and braced conditions. Forces resulting from wall inertia effects are expected to be relatively minor for non-gravity walls and may be ignored in estimating the seismic lateral earth pressure.
Bearing Capacity

Allowable bearing capacity of 1,500 psf will be used for the foundation design of the retaining walls, considering that the foundation will be at least 24 inches below the existing grade, 24 inches in width, and be supported on a minimum of 24 inches of engineered fill.

This value will be increased by one-third when considering loads of short duration, such as those imposed by wind or seismic forces. An allowable coefficient of friction value of 0.35 between the base of the footings and engineered fill can be used for sliding resistance using the dead load forces.

Tiebacks

The soft clayey/silty soils extend to El -35 feet, NAVD88. Therefore, it may be required to extend the retaining wall below this soft layer to secure the toe of the wall or for the foundation to reach sufficient axial capacity.

Tiebacks, if required, should be designed to derive their load carrying capacity from the soil behind the active wedge behind the wall. This wedge is defined by a plane drawn at approximately 61 degrees above horizontal from the bottom of the wall (i.e., the non-retained ground elevation). Tiebacks should have a minimum unbonded length of 10 and 15 feet for bars and strands, respectively. All tiebacks should have a minimum bonded length of 15 feet and be spaced at least 4 feet on center, with the bond zone beginning at least 5 feet behind the failure plane as defined above. The center of the bonded zone will be at least 15 feet below ground. Prior to installation of tiebacks, the contractor should verify site conditions so there is no conflict with existing utilities, foundations, and/or other subsurface structures. Tiebacks will be located so they are not within 3 feet of existing utilities if gravity-grouted or 5 feet of existing utilities if pressure-grouted.

The tieback ultimate capacity can be expected to range from approximately 0.6 to 1.4 kips per square foot for pressure grouted anchors, when grouted in silty/clayey material. However, these values are highly dependent on contractor methodology, and a factor of safety of at least 2.0 is recommended by the Federal Highway Administration (FHWA, 1999). Additionally, since the bonded length of the tiebacks are expected to mainly be located within the soft silty/clayey soils, the long-term creep of the tiebacks should also be investigated in the next design stage.

Pile Foundations

For preliminary purposes, axial capacities for 24-inch CISS piles were calculated using the Apile software (Ensoft, 2019a) for static condition. The soil profile used in the pile analysis was based on the existing nearby borings. Appendix B presents ultimate axial capacities for static condition. If liquefaction is later determined to be likely at this location, piles are likely to tip at approximately El -35 feet, NAVD88, to overcome downdrag effects.
Preliminary lateral capacities for the CISS piles were calculated using the Lpile software (Ensoft, 2019b). Appendix B presents lateral capacities of piles based on deflections of 0.25, 0.5, and 1.0 inch for pinned head connections. The estimated lateral capacities presented are for single piles and do not consider a reduction for group action.

11.4 Structural Alternative Analysis

The overall combined length of the proposed walls is approximately 790 feet and is composed of three segments:

- Segment 1 is called Gap Wall, with an approximate length of 250 feet;
- Segment 2 is called GDB C4 Wall, with an approximate length of 280 feet; and
- Segment 3 is called Tie-back Wall, with an approximate length of 260 feet.

11.4.1 Segment 1 – Gap Wall

Segment 1 joins the existing Wall Q and the proposed reconstructed GDB C4 Wall. Two alternatives are being evaluated. Under Alternative 1, it is proposed to use a Caltrans Type 1 retaining wall with deep foundation. To keep the southbound SR-710 exit ramp open to traffic during construction, this alternative will need temporary shoring. Under Alternative 2, the use of sheet pile wall with tieback is proposed. This alternative eliminates the need for a temporary shoring system, reducing construction duration. Appendix C includes Segment 1 structural alternatives.

11.4.2 Segment 2 – GDB C4 Wall

The existing C4 Wall at Segment 2 was constructed as part of the GDB project. The elevation of the proposed tracks will require additional excavation at the toe of the existing C4 Wall. Therefore, this wall needs to be altered to support the new elevation difference. The main challenge for this wall segment is the underground utilities crossing the wall plane.

Two alternatives are proposed for Segment 2. Alternative 1 proposes to replace the existing wall with a new Caltrans Type 1 wall with a deeper foundation. It will be challenging to avoid the existing underground utilities, which may result in uneven spacing between the piles. Alternative 2 proposes to use a combination of king pile/sheet pile wall system with tiebacks. Similar to Alternative 1, underground utilities need to be avoided to install the king piles. This may result in special design of the king pile/sheet pile system. Appendix C includes Segment 2 structural alternatives.
11.4.3 Segment 3 – Tieback Wall

The Segment 3 proposed wall alignment runs adjacent to the existing southbound SR-710 exit ramp bridge column at Bent 5. Two alternatives are proposed for the Segment 3 wall. Alternative 1 proposes a combination of king pile-sheet pile wall system with tiebacks passing underneath the existing bridge pile cap at Bent 5. Alternative 2 proposes to use king pile-sheet pile wall system with tiebacks and walers to avoid installing tiebacks underneath the existing bridge pile cap. The impact of the new wall on the existing bridge pile cap and piles will be evaluated. Appendix C includes Segment 3 structural alternatives.

11.5 Permitting

Table 11-3 provides a list of potential agencies that may require permits for the SR-710 walls.

<table>
<thead>
<tr>
<th>Permit</th>
<th>Agency with Jurisdiction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Caltrans Encroachment Permit</td>
<td>Caltrans</td>
</tr>
<tr>
<td>Division Land Use Permit</td>
<td>POLB</td>
</tr>
<tr>
<td>Harbor Development Permit</td>
<td>POLB</td>
</tr>
<tr>
<td>Fire Access Permit</td>
<td>COLB</td>
</tr>
<tr>
<td>Grading Permit</td>
<td>COLB</td>
</tr>
<tr>
<td>Building Permit</td>
<td>COLB</td>
</tr>
</tbody>
</table>

11.6 Record Documents

Table 11-4 summarizes the record documents reviewed for the structural analysis.

<table>
<thead>
<tr>
<th>Number</th>
<th>Record Document</th>
<th>Date</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>GDB C4 Wall As-built drawings; Appendix D</td>
<td>10/24/2018</td>
<td>POLB</td>
</tr>
<tr>
<td>2</td>
<td>GDB C4 Wall Calculation and Independent Checker Calcs packages</td>
<td>10/12/2016</td>
<td>POLB</td>
</tr>
<tr>
<td>3</td>
<td>GDB C4 Wall Foundation Report</td>
<td>09/27/2016</td>
<td>POLB</td>
</tr>
</tbody>
</table>
Table 11-4. Record Documents Reviewed

<table>
<thead>
<tr>
<th>Number</th>
<th>Record Document</th>
<th>Date</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>SR-710 Ramp As-built Drawings</td>
<td>06/10/1994</td>
<td>POLB</td>
</tr>
</tbody>
</table>
12.0 Crescent Warehouse Alteration

The existing Crescent Warehouse was originally built circa 1950. The building is located at Pico Avenue, extending over Berths D52, D53, and D54. Refer to Figure 3-1 for the Crescent Warehouse location.

Prior to construction of the existing warehouse, a pile-supported wharf extending over open water had been constructed for boat access. The wharf structure included a bulkhead adjacent to the channel and a pier with a series of piles with various lengths, and extended approximately 60 to 70 feet inland from the face of the bulkhead. This wharf was subsequently infilled, a deadman anchored bulkhead was constructed, and the wharf was capped with concrete and abandoned in place. This became the new “land” area over which a portion of the current warehouse was constructed. Therefore, various portions of the warehouse are supported on the buried wharf, above the deadman anchoring system, or on the older fill and alluvial soils.

The review of the available drawings, dated 1954, indicated that a subsidence/settlement remediation program was performed on the existing warehouse foundation system, which included raising the foundations up to 6 inches at some locations.

The building has an irregular shape, with six unequal, variable sides. The building’s longest side is approximately 710 feet, with a width that varies from 95 to 360 feet. Figure 12-1 shows a plan view for the location of the warehouse. The warehouse is composed of steel trusses supported on concrete and steel columns. The building is surrounded by concrete walls that have several door and window openings. The roof is composed of steel sheets supported on steel purlins. All building columns are supported on either concrete or timber piles, and the wall is supported on strip footing.

The Program proposes eight new tracks to run through Pico Avenue. This will require realigning the existing Pico Avenue roadway towards the building. To accommodate the width of the proposed widened Pico Avenue, part of the existing Crescent Warehouse will be altered with a new north wall. Figure 12-2 provides an aerial view of the existing warehouse, showing the altered area.

A site visit was conducted on December 12, 2019, to identify the above-ground constraints and potential conflicts. Refer to the Pier B Site Investigation Report 1-2-3 (HDR, 2019) for further details.
Figure 12-1. Existing Crescent Warehouse
12.1 Basis of Design

12.1.1 Applicable Codes, Standards, Guidelines, and Structures and Architectural Considerations

The Program will be designed and constructed in accordance with the latest version of the following codes and standards:

- CBC (2019)
- Requirements for Structural Concrete (ACI 318-14)
- Specification for Structural Steel Buildings (AISC 360-16)
- Seismic Provisions for Structural Steel Buildings (AISC 341-16)
• Minimum Design Loads and Associated Criteria for Buildings and Other Structures with Supplement No. 1 (ASCE 7-16)
• Structural Welding Code – Steel (AWS D1.1/D1.1M-15)
• Structural Welding Code – Seismic Supplement (AWS D1.8/D1.8M -2016)
• Structural Welding Code – Sheet Steel (AWS D1.3/D1.3M-08)
• Structural Welding Code – Reinforcing Steel Including Metal Inserts and Connections in Reinforced Concrete Construction (D1.4/D1.4M- 2017)
• CEC
• CEN
• CFC
• CGB Standard Code
• CMC
• CalOSHA General Industry Orders
• COLB Municipal Code
• NFPA

12.1.2 Materials

Existing Structure

• Concrete: Minimum compressive strength for reinforced concrete walls, grade beams, foundation = 3,000 PSI
• Reinforcing Steel: f_y = 20,000 PSI
• Structural Steel: f_y = 20,000 PSI

Proposed Wall

• Concrete: Minimum compressive strength at 28 days for CIP reinforced concrete walls = 4,000 PSI
• Reinforcing Steel: Will conform to the requirements of ASTM A706, Grade 60; all spiral reinforcement will conform to the requirements of ASTM A82
• Structural Steel: All structural steel will conform to the requirement of ASTM A709, Grade 50
• CISS: Grade A252
12.1.3 Loads and Load Combinations

- Dead Load:
  - Concrete = 150 pcf
  - Steel = 490 pcf
- Roof Dead Load: 8.8 psf
- Wind Load: Based on ASCE 7-16
- Seismic Load: For the proposed wall, EQ load is based on ASCE 7-16; design EQ is 2/3 of probability of exceedance 2 percent in 50 years. For the existing building, EQ load is based on the 2016 Existing Building Code probability of exceedance 20 percent in 50 years.
- Tsunami Hazard Assessment Report (M&N, 2007) stated that for the bulk of the area within POLB, no water overtopping is expected; therefore, tsunami loads are not considered in the design.

12.1.4 Sea Level Rise

The CRP prepared by POLB (2016) reported three scenarios for overtopping and inundation with and without surge for a 100-year return period. Table 12-1 summarizes the SLR for each scenario at the Crescent Warehouse. The SLR is not considered in the design, as agreed upon with POLB.

<table>
<thead>
<tr>
<th>100-year Storm Surge Scenario Included</th>
<th>SLR Scenario</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>16-inch</td>
</tr>
<tr>
<td>100-year Storm Surge Scenario Included</td>
<td>No</td>
</tr>
<tr>
<td>Crescent Warehouse Inundation</td>
<td>.a</td>
</tr>
</tbody>
</table>

*a Indicates no potential water accumulation

*b Indicates the structure will be below water surface without a direct hydraulic flow path

12.2 Existing Utilities and Right-of-Way Impact

12.2.1 Utilities and Potential Conflicts

The proposed end wall will be constructed to avoid existing underground utilities and existing deadman for the wharf bulkhead. Appendix A includes an exhibit showing the existing utilities at the Crescent Warehouse altered area.
12.2.2 Right-of-Way Impact

No right-of-way-impact is identified at this location.

12.3 Geotechnical Considerations

12.3.1 Subsurface Conditions

Subsurface materials at the site consist of up to approximately 12 feet of artificial fill over up to 45-foot-thick hydraulic fill over natural deposits. Artificial fill materials are shown to generally consist of loose to medium dense silty sand. Hydraulic fill generally consists of loose to very dense silty sand, clayey sand, and poorly graded sand with silt, with interbedded thin layers of low plasticity clays and silts. Natural deposits are shown to generally consist of medium dense to very dense silty sand, poorly graded sand with silt, well-graded sand with silt, and poorly graded sand, with interbedded thin layers of silts and clays. Appendix B provides boring logs and a cross section showing the nearby borings.

12.3.2 Liquefaction

The Program site is located within an area designated as potentially liquefiable by the California Geological Survey (1999). Based on the Program location being within a liquefaction zone and expected shallow groundwater depths, the potential for liquefaction exists at the site. The results of the preliminary liquefaction analysis indicated that approximately 7 inches of settlement may occur at the site following a significant seismic event. This evaluation will be updated during final design once HDR’s geotechnical investigation is completed.

12.3.3 Compressible Soils

Static compression is a primary consideration for this Program due to the potential for differential settlements between newly constructed elements and existing ones. Soft clay layers were encountered within the upper 50 feet and are considered to be compressible when subjected to additional loads such as moderate to heavy foundation loads and/or additional fill soils. Static settlement will be evaluated during the final design once HDR’s geotechnical investigation is completed.

12.3.4 Corrosion

Based on the corrosion test results by CH2MHiIl (2015), the subsurface soils in the upper 10 feet of the site are considered to be corrosive towards buried concrete and ferrous metals. Corrosion measures to protect steel reinforcement will be implemented.
12.3.5 Preliminary Geotechnical Recommendations

Seismic Design Spectra

ARS curves were developed for two boundary site classes (C/D and D/E) and a return period of 2,475 years using USGS’s (2020) unified hazard tool. Boundary of Site Class C/D with a $V_{s30}$ of 360 m/s was representative of a firm ground, resulting in higher spectral accelerations at lower spectral periods. Boundary of Site Class D/E with a $V_{s30}$ of 180 m/s was selected for the soft ground, resulting in higher spectral accelerations at higher periods. The preliminary ARS for the site was selected as the envelope of these two ARS curves increased by 20 percent to account for softer than assumed soil profiles and the impact from the Wilmington Fault not considered in the USGS fault models. Appendix B shows the preliminary ARS curve for the return period of 2,475 years.

Lateral Earth Pressure

Appendix B provides EFP values for the preliminary design of earth-retaining structures at the site using engineered fill. The EFP concept is commonly used in the estimation of the lateral earth pressure that a retaining wall or shoring system will be required to resist. EFP is expressed as the unit weight of a fluid (in pcf) that would generate a hydrostatic pressure equal to the anticipated lateral earth pressure at a given depth. This horizontal pressure is applied to a vertical plane extending up from the heel of the wall base, and the weight of soil above the wall heel is included as part of the wall weight. A soil unit weight of 120 pcf will be used for calculating the weight of the soil over a structure. The lateral earth pressures provided in Appendix B were estimated for granular engineered fill behind the wall.

Proposed Wall Foundation

Based on subsurface conditions (both soil and existing foundations/wharf) at the Program site, various options could be considered for the foundation of the proposed wall. Descriptions of potential alternatives for wall support at the warehouse are provided below.

Grade Beams

This alternative would use grade beams connected to the existing pile foundations that will transfer vertical loading (at minimum) and potentially some or all lateral loading onto the existing piles. The wall’s vertical load may be partially or fully offset from these pile caps by the removal of the tributary area of the building demolished. In this scenario, the existing pile system should be checked for lateral load capacity. If found inadequate, micro-piles or other pile solutions could be constructed to transfer the lateral load to the deeper subsurface soils.
Shallow Foundations

This alternative utilizes shallow foundations under the proposed wall. This eliminates load transfer from the wall to the existing structure foundation elements. In this alternative, shear keys shall be constructed beneath the wall foundation for better transfer of lateral loads to the subsurface soils, if needed. Because this alternative directly provides new vertical loading onto the soil, substantial static settlements are anticipated. While the existing available data does not allow for precise calculation of these settlement values, it is anticipated that settlements may be in the range of 1 to 4 inches based on loading and prior building settlement, which would manifest as a differential settlement between the proposed wall and the existing warehouse.

Deep Foundations

In this alternative, which is selected as the recommended option, the proposed wall will consist of two segments. The segment of the wall that is not above the existing wharf will be supported on pile foundations, and the other portion of the wall above the existing wharf will be constructed on a cantilever grade beam connected to the proposed wall foundation. This segment of the wall will perform as a filler and will not transfer shear loads from the superstructure to the foundation.

Based on preliminary evaluations, piles are expected to have tip elevations at approximately EL - 60 feet, NAVD88. These piles may be installed using either drilled (casing) or driven methods. The expected axial loading is relatively minor, but these tip elevations will ensure mitigation of seismic downdrag loading. Pile design is likely to be controlled by lateral loading. Appendix B presents preliminary lateral capacities of piles based on deflections of 0.25, 0.5, and 1.0 inch for pinned head connections for steel pipe piles with diameters of 24 inches and 36 inches.

12.4 Architectural Considerations

12.4.1 Design Code Compliance

This Program will implement code and design standards listed in Section 12.1.1, Applicable Codes, Standards, Guidelines, and Structures and Architectural Considerations.

12.4.2 Accessibility

The site and building will be designed for complete accessibility to meet the minimum requirements of CBC Chapter 11 (and other chapters) and the ADA. This includes providing accessible toilet, parking space, and path of travel to the building entrances.
12.4.3 Design Drivers

Functional Efficacy and Efficiency

The design intent is to enhance the use of the site and building layout to allow better workflows and maximize space usage.

It is also the intent not to interrupt the usage of the building. Considerations will be made for the phasing and construction sequence.

Workplace Quality and Safety

To ensure the quality and safety of this warehouse, the building will be designed to meet current code. Also, it is the intent to maximize the quality of the workplace environment concerning general comfort, lighting, and sightline.

Aesthetics

The design will be cost effective, durable, and low maintenance, and it will respectfully enhance the building while maintaining its architectural characteristics.

12.4.4 Building Envelope

Building Mass

The overall building mass will shrink to accommodate the west shift of Pico Avenue. The east façade has to move back by three bays. The existing gross building area is approximately 142,680 square feet, and the proposed gross building area will be 125,920 square feet. The new east façade will be longer compared to the original one due to a tapered building geometry. However, the new façade will maintain the architectural language used to give a coherent look to the exterior. The existing façade is made of reinforced concrete walls with roll-up doors and punch windows. In the new façade, the roll-up doors will be replaced by a storefront system to bring more natural light into the building. The exterior lighting used on this new façade will have to comply with 2019 Building Energy Efficiency Standards, Title 24.
12.4.5 Plan and Program

Functional Plan Requirements

Storage Area

Despite the shrinkage of the building mass, in an effort to gain maximum footage for storage space, the abandoned office area will be demolished. The net storage area overall will change from 136,514 to 121,260 square feet.

Toilets

This building currently has four toilets, two of which are located in abandoned office areas. The remaining two are located in the middle section. All can be accessed from the exterior. As the building footprint shrinks, the toilet in the east office area will be removed. To comply with the minimum number of required plumbing fixtures of the 2019 CBC, at least three unisex toilets need to be provided. One of the toilets needs to be ADA compliant. The existing toilets will have to be altered to meet these requirements.

Support Spaces/Utilities

To reduce the building footprint on the east side, utilities and support spaces will have to be relocated to ensure a fully functioning building. The phasing recommendation is to relocate the utilities before construction starts to allow the continuation of warehouse operation. Two electric boxes, switch boxes, and main fire risers will be relocated.

12.4.6 Interior Finishes

Built Interior Environment Concept

The interior intends to address maintainability for the use of the warehouse. The materials chosen will be both durable and easily cleanable.

Conceptual Interior Finishes

The interior finishes are to be determined.

12.5 Structural Alternative Analysis

The new added tracks along Pico Avenue will require creating horizontal clearance at the location of the Crescent Warehouse. The edge of the road crosses at the end of the first bay; however, a 30-foot setback is proposed as there is an SCE electric line that will be crossing parallel to the proposed railway with a 15-foot offset. Therefore, a part of the warehouse is required to be altered,
specifically the first three bays. Figure 12-3 shows a plan for the proposed cut location with the 30-foot setback line.

The building is eligible for historic registry, and no decision has yet been made in this regard as it is still pending the Environmental Impact Report to be published. Therefore, the impact of a historic building classification will be evaluated after this decision is made.

\textit{Figure 12-3. Existing Crescent Warehouse Proposed Alteration Set Back}

12.5.1 Existing Building Evaluation due to Alteration

The main lateral resisting system for the existing warehouse is reinforced concrete shear walls. A new wall will be added at the cut location to provide sufficient lateral seismic resisting stiffness. The new wall is to be designed to current code requirements, including the demand and design of the structural components based on CBC (2019). However, for the existing building seismic evaluation, COLB Municipal Code (Section 18.49.010) adopts the California Existing Building Code Part 10. Existing Building Code clearly specifies that the provisions of CBC Sections 317 through 323 may be adopted by a local jurisdiction for EQ evaluation and design for retrofit of existing building. CBC Section 317 criteria need to be evaluated for the existing building after alteration.
Preliminary evaluation was performed to determine the forces in the existing building shear walls with and without alteration. The shear walls in the longitudinal direction are reduced in length due to the alteration by approximately 20 percent. See Figure 12-4 and Table 12-2, which summarize the preliminary evaluation findings according to CBC Section 317 requirements. The 10 percent increase in forces with and without alteration requirement is not satisfied. The increase in shear wall forces due to the alteration ranges between 18 and 24 percent (see Table 12-3). Due to this increase, further evaluation is required according to CBC Section 317. CBC Section 317 refers to Section 319 for existing building seismic evaluation, which refers to CBC Section 320. CBC Section 320 adopts the linear static and linear dynamic procedures of ASCE 41-17, which provides guidance for the seismic evaluation and retrofit of existing buildings. ASCE 41-17, Chapter 2, specifies that for existing buildings that need to satisfy Life Safety Performance, the structure needs to be checked against ground motions that have a probability of exceedance of 20 percent in 50 years, which is 225 years of return period.

Figure 12-4. Existing Crescent Warehouse Longitudinal Shear Walls

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Applicability</th>
<th>Comments</th>
</tr>
</thead>
</table>
| 1 – Total Construction Cost of new structure less than 25% of construction cost for replacement of existing building | Altered Area = 16,400 ft²  
Total Building Area = 141,072 ft²  
% Altered = 11.6 < 25% | Criteria is met |
| 2 – Change in Risk Category | No change in risk category | Criteria is met |
Table 12-2. CBC Section 317 Requirements for Evaluating Building Alteration

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Applicability</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>3 – Seismic Forces increase to any structural component less than 10%</td>
<td>Force increase greater than 10%</td>
<td>Criteria is not met</td>
</tr>
<tr>
<td>4 – Structural elements need repair where damage has reduced the lateral-load-resisting capacity of the structural system by more than 10%</td>
<td>Does not apply; not aware of any damage to any of the warehouse structural components</td>
<td>Criteria is met</td>
</tr>
<tr>
<td>5 – Changes in live or dead load increase story shear by more than 10%</td>
<td>Does not apply</td>
<td>Criteria is met</td>
</tr>
</tbody>
</table>

Notes: ft² = square feet

Table 12-3. Longitudinal Shear Wall Forces Summary with and without Alteration

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Existing Condition</th>
<th>Altered Walls Condition</th>
<th>% Change</th>
</tr>
</thead>
<tbody>
<tr>
<td>Waterside Shear Wall</td>
<td>3.77 k/ft</td>
<td>4.47 k/ft</td>
<td>18.6% &gt; 10%</td>
</tr>
<tr>
<td>Landside Shear Wall</td>
<td>4.33 k/ft</td>
<td>5.40 k/ft</td>
<td>24.8% &gt; 10%</td>
</tr>
</tbody>
</table>

Notes: k/ft = kilopounds per feet

The presented preliminary results were based on simplified calculations without utilizing a computer model. Further detailed analyses will be performed in the 30% Design to investigate the seismic behavior of the altered building to evaluate the seismic demands change in the structural elements. This will include a 3D SAP2000 model that includes building modeling and foundation components for both the existing and altered buildings.

12.5.2 Proposed Wall Alternatives

The proposed wall will be designed as two parts: the wall part, indicated as blue in the elevation (Figure 12-5), will be designed as a structural shear wall resisting all lateral loading; the other wall portion, indicated as green in the elevation (Figure 12-5), is designed as a nonstructural wall with special detailing with the shear wall and the roof truss. This can be achieved by providing an expansion joint between the two proposed walls. Figure 12-6 shows the section of the proposed wall and how it is proposed to connect to the existing steel trusses. Figure 12-7 shows the plan view of the proposed wall and how the proposed foundation system fits in between the existing anchors and deadman. The structural shear wall will not be connected to the existing building...
foundation system to avoid adding vertical and lateral loading to the existing building foundation system. Appendix C has a structural exhibit showing the proposed wall details.

There are several underground components, including deadman, ties, wharf slab, and sheet pile wall. Deep foundation will not be used for the new wall at the location of the underground wharf slab.

*Figure 12-5. Proposed End Wall Elevation*

*Figure 12-6. Proposed End Wall Section*
12.6 Permitting

Table 12-4 provides a list of potential agencies that may require permits.

Table 12-4. Permits List for Crescent Warehouse Alteration

<table>
<thead>
<tr>
<th>Permit</th>
<th>Agency with Jurisdiction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Building Permit</td>
<td>COLB</td>
</tr>
<tr>
<td>Division Land Use Permit</td>
<td>POLB</td>
</tr>
<tr>
<td>Harbor Development Permit</td>
<td>POLB</td>
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<tr>
<td>Coastal Development Permit</td>
<td>CCC</td>
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12.7 Record Documents

Table 12-5 summarizes the record documents reviewed for the structural analysis.

<table>
<thead>
<tr>
<th>Number</th>
<th>Record Document</th>
<th>Date</th>
<th>Source</th>
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<tbody>
<tr>
<td>1</td>
<td>Existing Crescent Warehouse Foundation Drawings; Appendix D</td>
<td>02/16/1948</td>
<td>COLB Building Department</td>
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<tr>
<td>2</td>
<td>Existing Crescent Warehouse Trusses Drawings; Appendix D</td>
<td>06/10/1948</td>
<td>COLB Building Department</td>
</tr>
<tr>
<td>3</td>
<td>Existing Crescent Warehouse Foundation Drawings; Appendix D</td>
<td>07/7/1941</td>
<td>COLB Building Department</td>
</tr>
<tr>
<td>4</td>
<td>Existing Crescent Warehouse Utilities (MEP) Drawings; Appendix D</td>
<td>03/5/1947</td>
<td>COLB Building Department</td>
</tr>
<tr>
<td>5</td>
<td>Existing Crescent Warehouse Offices Drawings; Appendix D</td>
<td>03/31/1954</td>
<td>COLB Building Department</td>
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<td>6</td>
<td>Existing Crescent Warehouse Elevation Drawings; Appendix D</td>
<td>03/7/1955</td>
<td>COLB Building Department</td>
</tr>
<tr>
<td>7</td>
<td>Existing Berths D52, D53, and D54 Drawings; Appendix D</td>
<td>03/5/1947</td>
<td>POLB</td>
</tr>
</tbody>
</table>

Notes: MEP = Mechanical/Electrical/Plumbing
13.0 References


County of Los Angeles, 2020, Administrative Manual, Department of Public Works, Geotechnical and Materials Engineering Division, S004.0, Seismic Earth Pressures on Retaining Walls, Dated January.


ATTACHMENT 7
POLB Adaptation Plan & Vulnerability Technical Memo
ADAPTATION PLAN & VULNERABILITY
February 15, 2022

CCC Comments:

i. Need a very flushed out adaptation plan for CCC approval at the time of application or require a plan as a condition of approval that:
   1. specifies triggers for different adaptation measures, and
   2. prioritizing nature-based adaptation measures.

ii. Need clarifications on how to implement that adaptation measures (process-oriented):
   1. Menu of engineering solutions
   2. Triggers for each application

iii. Clarify whether there is any part of this project more vulnerable which could have ripple effects.

Depending on future sea level rise and the future horizon on which it could occur, the Port of Long Beach could potentially experience seawater inundation on railroad tracks and other infrastructure on a somewhat regular basis late in the Project’s design life during extreme storm surge or king tides occurring (one to two times annually) in the southern California region. Rail infrastructure materials are not sensitive to damage as a result of short-term flooding. If rails are submerged, train movement will stop, but would be expected to resume quickly post-flood. When more regular flooding events occur in the future as a result of sea level rise, the POLB will implement engineering solutions to keep these assets dry and free from periodic seawater inundation as outlined in the Pier B ODRSF Sea Level Rise Technical Memo as well as the POLB’s 2016 Climate Adaptation and Coastal Resiliency Plan (CRP).

In general, nature-based adaptation strategies are not appropriate solutions in a highly industrial seaport such as the POLB. Maintaining the structural integrity and navigational safety of the various wharfs, channels and structures within the harbor requires the facilities to be hardened and armored. There are no opportunities for nature-based solutions such as beach nourishment, coastal wetland construction, off-shore oyster bed construction/restoration, etc., within the project footprint. However, the POLB remains open to nature-based strategies in other areas of the harbor if they can be designed and implemented to meet the structural and navigational requirements of the Port.

Based on modeling results, more periodic and predictable flooding impacts to the project footprint due to sea level rise would likely occur after the useful life of the project or after further redevelopment of the area due to market forces and the ever changing capacity and operational needs of the Port. Accordingly, it would be speculative to identify specific adaptation measures which would be implemented in the project footprint at this time. The Port of Long Beach rail network is identified as a critical operational system within the CRP. As the CRP is updated in
future years, the entire rail network at the Port will be assessed and appropriate adaptation strategies will be applied to the entire rail network, including the project footprint, to ensure business continuity. Potential adaptation strategies may include temporary or semi-permanent measures such as sand bags or tiger dams. Permanent barriers such as earthen berms, steel sheet pile walls, reinforced-concrete cantilevered walls, as well as seawall retrofits, additional drainage pathways, and/or embankments may also be considered, as appropriate.

The Port’s CRP assesses vulnerabilities in the POLB rail system as a whole and identifies specific areas of susceptible to inundation so adaptation measures can be developed to mitigate sea level rise impacts to rail operations. The entire POLB rail network as a system must be evaluated, not just areas within the project footprint, to ensure a comprehensive and holistic approach to assessing vulnerabilities and applying adaptation strategies. The Port’s CRP is the appropriate vehicle for this evaluation and future iterations of the Port’s CRP will conduct this holistic evaluation and develop appropriate adaptation strategies for the POLB rail network if necessary.
ATTACHMENT 8

POLB High Groundwater Table Technical Memo
HIGH GROUNDWATER TABLE
February 15, 2022

CCC Comments:
1. Describe to what extent is groundwater rise accounted for with SLR.
2. Is there a problem for corrosion?
3. Can you switch the materials?
4. Can pumping be implemented?

BACKGROUND

As the project was being developed, groundwater was considered in the draft Environmental Impact Report, section 3.1. “Groundwater contour maps from the Water Replenishment District of Southern California (MWD, 2007) indicate that inland production well pumping creates a northeasterly gradient towards a large pumping depression north of the Project Site.” “Site-specific groundwater conditions with the Project area are influenced by Long Beach Harbor to the south, the Dominguez Gap Sea Water Intrusion Barrier Project to the west, groundwater contamination cleanup within the general area, and production well pumping a few miles inland to the north and northeast.” (Draft EIR p. 3.1-5. Section 3.1). The EIR did not identify significant groundwater impacts due to the Project.

Four Preliminary Design Geotechnical Reports have been prepared for the Project, dated April 2021, including site-specific reports for the Dominguez Channel Bridge, Crescent Warehouse, and Pump Stations, and an overall report of the Project, and a Corrosion Study. These reports include site investigation on soil criteria and groundwater elevations, and make recommendations to the designers on seismic risk, corrosion potential, and soil properties to ensure the designers consider these conditions. Figure 1 shows the areas in blue within the Project footprint where the groundwater depth is currently less than 7” from the existing ground elevation as identified in the geotechnical borings completed for the preliminary design geotechnical reports.

Figure 1: Shallow Groundwater in Pier B ODRSF Program Footprint
It is widely known that groundwater table levels are relatively shallow throughout the POLB harbor district, so this concern is well understood and mitigated for during planning and construction by POLB planners and engineers.

Preliminary groundwater rise modeling is available to climate science practitioners via the latest CoSMoS-GW mapping tool available through the Our Coast Our Future (OCOF) website (see example in Figure 2 below), however a further, more in-depth analysis via this mapping tool would not provide significantly more valuable information to POLB staff in preparation of the design of the Project.

![Groundwater Hazard Map](image)

*Figure 2: Groundwater Hazard Map for Port of Long Beach Project Area per the CoSMoS-GW Model*

However, as sea levels rise in the future, there is a concern that groundwater levels would rise within the Project footprint. A rise in groundwater levels could result in the following long-term and temporary impacts:

- **Long-Term Impacts:**
  - Increased buoyancy within pipelines
  - Increased hydrostatic pressure on permanent structures
  - Increased maintenance requirements and maintenance costs
  - Increased groundwater intrusion within sewer and storm drain infrastructure
  - Increased corrosion

- **Temporary Construction Impacts:**
  - Increased construction costs, risks and challenges related to groundwater during construction
INCREASED BUOYANCY WITHIN PIPELINES & HYDROSTATIC PRESSURE ON PERMANENT STRUCTURES

Issue

Increased buoyancy within pipelines and hydrostatic pressure on permanent structures have the potential to impact existing and new or relocated utilities and structures with the Project footprint. When utilities have shallow ground cover there is a potential for groundwater to cause pipelines to float or thrust up through the soil due to increased buoyancy.

Mitigation

Generally, if a utility is installed at 3’ below ground or greater, the soil cover can counteract this potential for buoyancy. Utilities that cross the Pier B ODRSF Project are required to be installed at 5.5’ below the top of the railroad ties which means that utilities within the Pier B ODRSF Project Footprint will not be in danger of thrusting up through the soil. As part of the Program, utilities that may be impacted by rising groundwater will be identified so that POLB can implement an inspection program to identify any issues as early as possible and install anchors or other measures to counteract the increased buoyancy.

INCREASED HYDROSTATIC PRESSURE ON PERMANENT STRUCTURES

Issue

Increased hydrostatic pressure on structures within the Project footprint becomes a concern where the increased hydrostatic pressure results in a change in the loading characteristics that a structures design was based upon. This will primarily affect retaining walls or building structures below ground. Designing these structures for future groundwater conditions is cost prohibitive.

Mitigation

POLB will implement an increased inspection program for the structures within the Project footprint so that any impacts to the structures are identified early and mitigation measures can be implemented.

INCREASED MAINTENANCE AND COSTS

Issue

It is expected that maintenance needs and costs will increase where rising groundwater causes contamination of railroad ballast, degradation of railroad and roadway base and increased soil settlement resulting in increased pavement degradation (more potholes).

Mitigation

In order to mitigate the potential increased maintenance impact, the Pier B ODRSF Project has raised the elevation of the rail yard as high as possible while still providing a level grade for the rail yard operations and adequate vertical clearances under the Ocean Blvd., I-710/ Pico Ave. On-Off Ramps, Anaheim St. and Terminal Island Freeway Overpasses. In addition, POLB will
closely monitor the groundwater levels and implement a robust maintenance program for the rail yard and roadway improvements to ensure the increased maintenance due to rising groundwater will not impact POLB operations.

INCREASED GROUNDWATER INTRUSION WITHIN SEWER AND STORM DRAIN INFRASTRUCTURE

Issue

It is anticipated that groundwater intrusion within sewer and storm drain infrastructure could increase as a result of sea level rise.

Mitigation

Where sewer or storm drain utilities are proposed within the existing groundwater, these utilities are constructed with rubber gasketed joints to prevent groundwater intrusion. The Project will look into providing rubber gasketed joints on these utilities at the future anticipated groundwater depths to minimize any future ground water intrusion.

INCREASED CORROSION

Issue

Rising ground water due to an increase in the sea level will result in an increase in the amount of brackish, salty groundwater that comes into contact with steel and ductile iron pipelines as well as the steel within concrete structures. This could lead to an increase in the amount of corrosion that these pipelines and structures experience.

Mitigation

Existing pipelines within the project footprint are already designed to provide corrosion protection in the form of cathodic protection since the pipelines are constructed within a marine environment. There will be new steel casings proposed where utilities are crossing the proposed railroad tracks. Each casing will require cathodic protection and careful consideration to avoid increasing the potential for corrosion on adjacent pipelines or within the carrier pipe installed within the casing. The project will work with 3rd party utility companies and the public utility providers to ensure cathodic protection is included within the design of any new or relocated pipelines associated with the project. The project will also consider alternative pipe materials that are more resistant to corrosion such as PVC or HDPE where the project design requirements will allow. Corrosion protection experts are a part of the project design team and will be analyzing specific site constraints for the proposed buildings and structures, such as the Dominguez Bridge Widening or the POLB Sewer Lift Station. A corrosion analysis and protection recommendations technical memorandum will be prepared to document the best methods for protecting the various project elements from corrosion and the potential increased risk of corrosion from rising ground water.
INCREASED CONSTRUCTION COSTS, RISKS AND CHALLENGES RELATED TO GROUNDWATER DURING CONSTRUCTION (TEMPORARY IMPACTS)

Issue

With rising groundwater comes increased risks and costs associated with constructing within groundwater. POLB does not allow any active dewatering within the Harbor District, so any trenching or deep excavations will have to deal with groundwater throughout the duration of the construction. This becomes a challenge for getting compaction of backfill, keeping concrete dry as it cures, keeping workers safe, and any other activities that need to take place within the excavation.

Mitigation

One solution the Project is implementing to minimize these risks is to employ deep soil mixing before completing any deep excavations. Deep soil mixing helps to prevent the migration of groundwater through the area of the deep soil mixing so the excavation will not have to deal with groundwater continually entering the excavation. Groundwater levels will need to be monitored throughout the design and construction of the Project to ensure that any increased risks associated with rising groundwater are identified early and incorporated into the design.
ATTACHMENT 9
POLB Design Life Technical Memo
CCC Comments:
1. Need clarifications on 30, 40, 50-year design life. Does that take into consideration the amount of time the project is constructed, when does it start counting?
2. How old is the other rail infrastructure in the port?

DESIGN YEAR VS. DESIGN LIFE SPAN

The design year of an infrastructure project represents the year that the proposed infrastructure has been designed to support in terms of the projected infrastructure usage growth. The design life span for an infrastructure improvement represents the anticipated life span the infrastructure is designed to last based on typical conditions, uses and physical properties. The Pier B ODRSF Program EIR indicated a projected design year of 2035 to support the projected growth in container volume to be moved by on-dock rail within the port. This projected design year helped to establish the number of tracks to be constructed within the railyard and any roadway improvements required to deal with changes in traffic associated with the project. The design life span for the critical infrastructure improvements that are a part of this project varies with each type of infrastructure as indicated below:

- Bridge Structure – 75 Years
- Buildings – 50 Years
- Drainage Pipeline - 50 Years
- Sewer Pipeline – 50 Years
- Water Pipeline – 50 Years
- Mechanical/ Electrical Equipment – 50 Years
- Track – 25 Years
- Roadway Pavement – 20 Years

The program is planning to be under construction with various contracts from 2025 through 2032 and the majority of the infrastructure will be completed between 2030-2032. The design life span of the program infrastructure should be based upon the time the infrastructure construction is completed.

- Long-Term Impacts:
  - Design Life Span will be reduced due to SLR
DESIGN LIFE SPAN MAY BE REDUCED DUE TO SLR

Issue

Design life span of the infrastructure to be constructed by the Pier B ODRSF Program may be reduced by the projected sea level rise.

Mitigation

The design life span for bridge structures and buildings is based upon a design that takes into consideration the current seismic code requirements, existing ground and groundwater conditions and the proposed design loads. These design parameters are not impacted by sea level rise except where a rise in groundwater occurs. A rise in groundwater could increase the hydrostatic pressure that the structures are to be designed to and a mitigation measure for this has already been discussed in the rising groundwater impacts section.

For drainage, sewer, and water pipelines as well as the mechanical and electrical equipment within the proposed storm water pump station and sewer lift stations the design life span takes into consideration the material type, installation condition and the proposed design loads and pressures that the pipeline or equipment will operate under. Increased sea level rise could result in an increase in the groundwater levels within the project footprint which would result in increased corrosion that would reduce the design life of the pipeline and equipment. Mitigation measures to protect against the potential for increased corrosion have been discussed in the rising groundwater impacts section.

The proposed roadway pavement and track within the project limits is only projected to have a design life of 20-25 years respectively. Due to the nature of the POLB’s operation there will be continuous growth in projected container volumes and a need to continue to upgrade and improve the rail and roadway network within the port. As an example, all of the existing rail infrastructure within the project limits has been reconstructed within the past 20 years primarily as part of the Alameda Corridor project that was completed in the late 1990’s. This means that the roadway pavement and track infrastructure that will be constructed by the project will be reconstructed within the anticipated design life. Each time these elements of the proposed infrastructure are reconstructed the port will evaluate the impacts of SLR and design to the changed conditions at the time of the reconstruction.

AGE OF EXISTING INFRASTRUCTURE

In response to question 2, nearly all of the Port rail network has been built or re-built since 1990. Railroad track infrastructure can be maintained to last a long time, including re-profiling railroad tracks, adding ballast and replacing drainage. However, the Port complex tends to experience
major expansion efforts that require significant expansions to support continued growth. In the case of the Pier B Rail Yard, it will be replaced at or before its 30-year anniversary. Rough timetable is as follows:

- Pier B Rail Yard, rebuilt in 2000
- Alameda Corridor, constructed in 2002
- Triple Track at Ocean Blvd, completed in 2016
- Pier A on-dock rail yard, completed in 1997
- Pier J South, completed in 1993
- Pier T on-dock rail yard, completed in 2002
- Pier E on-dock rail yard, completed in 2021
- Pier G north on-dock rail yard, completed in 2012.