CENTRAL COAST LAYOVER FACILITY

Preliminary Geotechnical Design Report

San Luis Obispo, California

May 14, 2021

Prepared for:

LOSSAN RAIL CORRIDOR AGENCY
600 S. Main Street
Orange, CA 92868

Prepared by:

HDR Engineering, Inc.
3230 El Camino Real, Suite 200
Irvine, CA 92602

May 14, 2021

LOSSAN Rail Corridor Agency

600 S. Main Street

Orange, CA 92868

Subject: Preliminary Geotechnical Design Report

LOSSAN Central Coast Layover Facility Project

We are pleased to present this geotechnical design report summarizing the results of our subsurface exploration and engineering evaluations along with the recommendations for design and construction of the subject project.

If you have any questions regarding this report, please do not hesitate to contact us. We appreciate the opportunity to be of service.

Respectfully submitted,

HDR ENGINEERING, INC.

Manuel Guzman, PE
Engineer - Geotechnical

Mario Flores, EIT
Geotechnical Designer

Gary R. Goldman, PE, GE
Section Manager – Geotechnical

MG/MF/GRG

Table of Contents

List of Tables

List of Appendices

1.0 INTRODUCTION

1.1 PROJECT BACKGROUND

The proposed Central Coast Layover Facility (CCLF) Project (Project) is located at the northern end of the Los Angeles-San Diego-San Luis Obispo (LOSSAN) rail corridor in San Luis Obispo, California. The LOSSAN rail corridor is 351 miles in length and serves Metrolink and Coaster commuter trains, Amtrak intercity trains, and BNSF Railway and Union Pacific Railroad freight trains. The Project is initiated by the LOSSAN Rail Corridor Agency and proposes to relocate the existing Amtrak single-track layover facility, located directly across from the San Luis Obispo train station, which is the northernmost point of the Pacific Surfliner service. The new facility is proposed approximately ½-mile south of the existing facility within the Railroad District of San Luis Obispo, at the site of a former Southern Pacific roundhouse and locomotive maintenance facility that was built more than 100 years ago. After completion, it will facilitate maintenance of equipment and increase overnight layover and storage capacity at this railroad terminal. The Project site location is shown in Figure 1 in Appendix A.

The proposed improvements include extension or new construction of three layover tracks, each approximately 1,000 feet long; various operations, maintenance, and administrative office buildings totaling approximately 12,000 square feet; an approximately 10,000 square foot train wash facility, parking areas and access roads; and other ancillary improvements including equipment pads and poles. Additionally, a pedestrian trail is proposed along the western edge of the project. The locations of the proposed improvements are shown on Figure 2 in Appendix A.

1.2 PURPOSE AND SCOPE

HDR previously prepared a Preliminary Geotechnical Assessment Report (HDR, 2019) without subsurface exploration to support the site selection during a previous phase of the Project. The purpose of this preliminary geotechnical design report is to collect subsurface information at the site and provide geotechnical recommendations for the design of the proposed Project. The scope of work for this preliminary geotechnical design report included the following tasks:

- Review geotechnical maps and reports available online or in our in-house library that are relevant to the Site.
- Perform a Site reconnaissance to mark the proposed boring locations and contact Underground Service Alert (USA, also known as DigAlert) for utility clearance. Perform a geophysical survey to identify potential buried utilities and other detectable subsurface obstructions in the immediate vicinity of proposed boring locations prior to performing field exploration.
- Perform a subsurface exploration consisting of drilling, logging, and sampling of six (6) hollow-stem auger (HSA) borings to depths ranging between 5 and 50 feet below ground surface (bgs). One boring was converted to an infiltration test to measure in-situ water infiltration rates (see Section 2.2 for additional details).
- Perform geotechnical laboratory testing on selected soil samples.
- Perform geotechnical evaluation of the collected data.

- Prepare this preliminary geotechnical design report presenting our preliminary findings and geotechnical recommendations for the proposed improvements.

2.0 GEOTECHNICAL INVESTIGATION AND LABORATORY TESTING

2.1 SUBSURFACE EXPLORATION

Subsurface exploration consisted of advancing six (6) 8-inch diameter HSA borings to a maximum depth of approximately 50 feet bgs. The borings are located within a currently undeveloped parcel adjacent to the existing railroad right-of-way. Infiltration testing was performed in Boring A-21-004 to assess infiltration capabilities where basins are proposed. The approximate locations of the borings are shown in Figure 2 in Appendix A. Approximate boring coordinates, ground surface elevations, and depths explored are summarized in Table 2-1.

Table 2-1. Subsurface Exploration Information

Boring ID	Latitude	Longitude	Ground Surface Elevation (feet), NAVD 88	Exploration Depth (feet)
A-21-001	35.27167	-120.65460	247	26.3
A-21-002	35.27080	-120.65414	243	25.4
A-21-003	35.26984	-120.65373	239	50.7
A-21-004	35.26948	-120.65360	239	10.0
A-21-005	35.26886	-120.65319	237	25.7
A-21-006	35.26842	-120.65246	241	26.5

Notes:

- Information presented in this table is approximate.
- Ground surface elevations were obtained from Google Earth Pro™.

HDR conducted a Site reconnaissance on March 11, 2021 to evaluate the surface conditions and accessibility of the Site for field equipment and to mark the proposed boring locations. The borings were marked in the field by measuring the distance from existing Site features and by using a global positioning system (GPS). Subsequently, Underground Services Alert of Southern California (also known as DigAlert) and Union Pacific Railroad (UPRR) were contacted to identify subsurface utilities and obtain clearance for advancing borings at the Site. Additionally, an independent third-party geophysical subconsultant was used by HDR to clear the boring locations prior to drilling.

Borings were drilled on April 12, 2021 using a truck-mounted CME-75 drilling rig equipped with an 8-inch diameter HSA. Driven samples were collected at approximately 5-foot intervals within the HSA borings. Standard Penetration Tests (SPTs) were performed using a SPT sampler driven for a total penetration of 18 inches (or until practical refusal) into soil. Ring samples were collected using a Modified California (MC) sampler. Both samplers were driven using a 140-pound automatic hammer falling from a 30-inch height and the blow counts per 6 inches of penetration were recorded in the boring logs. The field sampling procedures were conducted in general accordance with ASTM Standard Test Methods D1586 and D3550 for SPT and split-barrel sampling of soil, respectively. In addition to driven samples, bulk samples were also collected from drill cuttings at selected borings.

The test borings were logged in the field by HDR geotechnical staff. Each soil sample collected was reviewed and described in general accordance with the Unified Soil Classification System (ASTM D2487). Soil samples were delivered to *AP Engineering and Testing* for laboratory testing.

After completion of drilling, the borings were backfilled with soil cuttings. Geotechnical boring logs are included in Appendix B. Note that the blow counts presented on the logs are actual field blow counts and have not been adjusted for the effects of overburden pressure, input driving energy, rod length, sampler correction, boring diameter, or other factors.

2.2 Infiltration Testing

Infiltration testing was performed within Boring A-21-004 in general accordance with the Deep Quick Infiltration Testing Methodology presented in the County of San Luis Obispo Post Construction Requirements Handbook (County of San Luis Obispo, 2017). Design flow rates, discharge volumes, and basin locations are not available at this time. The results of this test presented information only during this preliminary phase of the Project. Additional infiltration testing will likely be required during future design phases.

A 10-foot long section of 3-inch diameter perforated drain pipe with sock was installed in the borehole. The annular space was filled with a pea gravel filter pack.

Prior to performing the infiltration test, the test hole was pre-soaked by filling it with water. The hole did not completely drain within thirty minutes. The drop in water level was measured at approximately 30-minute intervals, refilling the hole to the original water depth after every reading. The test was performed for a total of 4 hours. Generally, the drop in water level was consistent throughout the test. The test data is presented in Appendix B and summarized in Table 2-2. The borehole was backfilled with soil cuttings after the testing was completed.

Table 2-2. Infiltration Test Data Summary

Test Location	Test Depth (feet)	Infiltration Rate (in/hr)	Soil Type
A-21-004	0-10	0.7	Clayey Sand with Gravel (upper 7.5 feet), fat Clay below

Based on the materials encountered and our observations during the testing, it is likely that most of the infiltration happened in the upper 5 feet where more granular and gravelly material was encountered. A reduction factor, which accounts for nonvertical flow, was applied to raw percolation rates based on the Los Angeles County Percolation test Procedure (2014), however, raw data is presented in Appendix B and the designer may choose to apply other reduction factors. A factor of safety has not been applied to the infiltration rate. A factor of safety of at least 2.0 is recommended by Caltrans (2011a) and others. The designer may apply an appropriate factor of safety based on the selected method of infiltration.

Our scope of work was limited to testing, and does not include evaluation of the general suitability of the project site for the infiltration system, evaluation of the storage capacity, nor actual design of the infiltration system. The actual infiltration rate may vary from the values reported herein. The design elevation and size of the proposed infiltration systems should account for the expected variability in infiltration rates. The proposed storm water management system design should be performed by the project Civil Engineer. Additional infiltration basin construction and design recommendations are provided in Section 6.5

2.3 GEOTECHNICAL LABORATORY TESTING

Laboratory tests were performed on selected soil samples to determine the geotechnical engineering properties of subsurface materials. The following laboratory tests were performed:

- In-situ moisture content and density;
- Grain-size distribution;
- Atterberg limits;
- Direct Shear;
- Consolidation;
- Sand Equivalent;
- Expansion Index;
- Laboratory Compaction (maximum dry density and optimum moisture content);
- R-Value; and
- Corrosivity (soluble sulfate contents, chloride, pH, and resistivity).

All laboratory tests were performed in general accordance with ASTM procedures, except corrosivity tests, which were performed in accordance with the Caltrans procedures. Results of the laboratory tests are presented in Appendix C and summarized in Table C-1.

3.0 GEOLOGY AND FAULTING

3.1 REGIONAL GEOLOGY

The Project site is located within the southern portion of the Coastal Ranges geomorphic province of California. The Coastal Ranges are characterized by a series of low mountain ranges and valleys that trend northwest, subparallel to the San Andreas Fault. Generally, the ranges consist of elevations ranging from about 2,000 to 4,000 feet, and with the highest reaching 6,000 feet above sea level (Fuller et al. 2015). The southern Coast Ranges are mainly comprised of sedimentary, volcanic, metavolcanics, melanges of serpentinite, and igneous rock (Wiegers, 2010).

3.2 SITE GEOLOGY

Based on the geologic map by Wiegers (2010), the Project site is generally located on surficial deposits consisting of Mélange of Franciscan Complex (KJfm) of Cretaceous to Jurassic age. Other geologic units located in the vicinity of the Site include Young Alluvial Flood-Plain Deposits, Unit 1 (Qya1) of Holocene to late Pleistocene age.

The Mélange unit consists of fragmented rock masses embedded in a penetratively sheared matrix of argillite and crushed metasandstone. The large block masses include high grade blue schist, greenstone, greywacke, and chert.

The Young Alluvial Flood-Plain Deposits are described as unconsolidated sand, silt, and clay-bearing alluvium deposited on floodplains and along valley floors. Surficial soils may also contain fill and other materials from previous construction activity at the Project site. A regional geologic map is presented in Figure 3 in Appendix A.

Although not shown in the available geology maps, fill was encountered in all the borings presumably placed during previous construction activities.

3.3 FAULTING AND SEISMICITY

Our review of California Earthquake Hazards Zone Application (EQ Zapp) available online by California Geological Survey (CGS, 2021) and the USGS quaternary fault database (USGS, 2020) indicates that the Project site is not underlain by known active or potentially active faults, nor does the site lie within a Special Studies Zone.

The principal seismic hazard that could affect the Project site is ground shaking resulting from an earthquake occurring along one of several major active or potentially active faults in the vicinity of the Project site.Table 3-1 lists faults with a risk contribution greater than 1 percent, along with pertinent data such as fault type, distance to fault, and maximum magnitude. Other nearby faults are shown in Figure 4.

Table 3-1. Contributing Faults

Fault Name	Distance R_{rup} (miles)	Moment Magnitude (Mw)	Fault Type
Los Osos 2011	3.2	7.3	Reverse
Oceanic – West Huasna	3.5	6.8	Reverse
San Luis Range	6.1	6.8	Reverse
Rinconada	8.5	6.7	Strike-slip
Hosgri	15.0	7.5	Strike-slip
San Andreas (Cholame)	36.3	8.1	Strike-slip

Note:

Listed faults were derived from United States Geologic Survey (USGS) Deaggregation online tool and lists faults with a risk contribution greater than 1 percent of the total seismic risk. Site Class D was assumed and using USGS Dynamic 2014 dataset (V4.2.0) with a 2,475-year return period. See USGS (2021c) for details

4.0 PRELIMINARY GEOTECHNICAL FINDINGS

4.1 EXISTING SURFACE CONDITIONS

Based on the existing data (Google Earth Pro^{TM}, 2019), the existing ground surface elevations at the Project site range from approximately 237 feet above North American Vertical Datum 88 (NAVD88) to 248 feet NAVD88. The existing site developments include railroad tracks, paved areas, sparse vegetation, and minor structures. There are also remnants of previously demolished structures at the site including concrete foundations and slabs which are likely from the former Southern Pacific roundhouse and locomotive maintenance facility. Based on the site topography, surface water runoff appears to drain toward the southwest. Residential and commercial development exists in the vicinity of the Project site.

4.2 SUBSURFACE EARTH MATERIALS

Subsurface conditions at the Project site were observed to consist of about up to 7.5 feet of fill over native material.

The fill depths varied throughout the site and was noted to extend to a depth of 3 feet in most borings except Boring A-21-001, A-21-002, and A-21-005 where it was encountered at 5, 7.5, and 7.5 feet respectively. The fill was generally consisted of clayey sand with gravel with the exception at Boring A-21-006 which consisted of silty sand with gravel. Thicker fill layers may exist within unexplored areas throughout the site.

The native materials below the fill generally consisted of lean clay to depths of about 25 to 45 feet bgs. Thin layers (2 to 8 feet thick) of medium stiff to hard fat clay were encountered in four of the borings between depths of about 5 to 15 feet bgs. Thin layers (about 3 to 8 feet) of clayey sand and silt were also noted at various depths throughout the borings. In Boring A-21-005, a thin layer (about 3 feet) of clayey gravel was encountered at a depth of 5 feet. The consistency of the clayey and silty native material from soft to hard (generally hard below a depth of 15 feet) throughout the borings. When encountered at shallow depths (upper 5 to 10 feet), the consistency of the granular soils was medium dense, and very dense when encountered at depth (greater than 20 feet bgs).

4.3 ENGINEERING PROPERTIES OF SUBSURFACE MATERIALS

4.3.1 Shear Strength

Three direct shear tests were performed on the clayey sand to sandy clay material at depths between 5 and 15 feet bgs. Based on the direct shear test results, the cohesion intercept (c) and friction angle (ϕ) representing the effective ultimate shear strength of the tested soil ranged between 200 and 450 pounds per square foot (psf), and 25 and 36 degrees, respectively. Based on the field and laboratory test results and soil types, generalized shear strength parameters and unit weights selected for preliminary design are presented in Table 4-1. The test results are presented in Appendix C.

Table 4-1. Soil Design Parameters

Generalized Soil Type	Depth Below Grade (feet)	Total Unit Weight (pcf)	Friction Angle (degrees) (1)	Undrained Shear Strength (psf) (1)
Clayey Sand with Gravel	0-5	120	30	--
Medium Stiff Clay	5-13	130	--	2,000
Hard Clay/Silt	13-50	130	--	3,000-5,000

4.3.2 Density and Compaction

The measured dry density in the upper 5 feet (driven samples taken at a depth of 5 feet) ranged between approximately 99 pcf and 107 pcf with an average of 102 pcf. The water content of these samples varied between approximately 10 and 22 percent with an average of approximately 15 percent.

Using the laboratory maximum dry density values obtained based on the ASTM Test Method ASTM D1557, the estimated maximum dry density of the existing near-surface subgrade materials (upper 5 feet) ranges from approximately 123 to 132 pcf with an average of 129 pcf. The optimum moisture content ranged from about 9 to 10 percent with an average of 9.4 percent.

4.3.3 Expansive Soils

Expansion index (EI) testing was conducted at three locations. The EI test represents the tendency of soils to expand when wetted or contract when dried. Test results indicated that the soil within the upper 5 feet had EI values ranging between 0 and 57 corresponding to very low to medium expansion potential. It should be noted that EI testing was performed on the bulk samples collected within the upper 5 feet. Other soil types encountered at depths greater than 5 feet may exhibit higher expansion potential. Additional testing should be performed during final design to evaluate the expansion potential of the soils below proposed footings at the site.

4.3.4 Corrosion Potential

Analytical testing were performed on soil samples at four locations to evaluate the potential for corrosion to concrete and ferrous metals. Caltrans Corrosion Guidelines (2018a) define corrosive soils as materials in which any of the following conditions exist:

- Chloride content greater than 500 parts per million (ppm);
- Soluble sulfate content greater than 1,500 ppm; or
- pH of 5.5 or less.

Based on the corrosion test results presented in Table 4-2 and using the Caltrans criteria, the subsurface soils at the Project site generally have a low corrosion potential to buried concrete materials, except corrosive material was found in Boring A-21-002. Table 4-2 also presents corrosion potential and sulfate class based on the recommendations of National Association of Corrosion Engineers (NACE, 1984) and American Concrete Institute (2019), respectively. Using the NACE criteria, the subsurface soils are generally considered moderately to severely corrosive to buried ferrous metals.

Corrosion test results reported in Table 4-2 are only meant to be utilized as a screening process for indication of soil corrosivity. For detailed evaluation of corrosion potential at the Project site, a corrosion engineer should be consulted. HDR provides corrosion engineering services for both testing and design of corrosion resistant structures, and services can be provided upon request. The corrosion test results are included in Appendix C.

Table 4-2. Summary of Corrosion Test Results

Boring ID	Sample Depth (feet)	pH	Minimum Resistivity (ohm-cm)	Sulfates [Class[1]] (ppm)	Chlorides (ppm)	Corrosivity (Caltrans)	Corrosivity (NACE)
A-21-001	0-5	8.7	5,576	34	16	Non-Corrosive	Moderately Corrosive
A-21-002	7.5	8.4	575	130	538	Corrosive	Severely Corrosive
A-21-003	25	9.6	2,330	20	19	Non-Corrosive	Moderately Corrosive
A-21-005	15	9.2	2,279	25	15	Non-Corrosive	Moderately Corrosive

Notes:

1. Sulfate Class S_0 for all tested material per recommendations of American Concrete Institute (2019).
2. ohm-cm = ohm centimeters; ppm = parts per million

4.3.5 Pavement Support (R-Value)

Two bulk samples were tested for R-value. The samples taken at A-21-004 and A-21-006 indicated R-values of 18 and 76, respectively. It should be noted that material encountered at A-21-006 had lower plasticity and higher gravel content compared to the rest of the borings. The test result at A-21-004 may be more representative of the site conditions.

4.3.6 Compressible Soils

Clay layers encountered in the upper 50 feet were generally medium stiff to hard in consistency with the exception of a thin layer (about 3 feet thick) in Boring A-21-006 where soft clay was encountered at a depth of about 7.5 feet. Two consolidation tests were conducted in the upper 10 feet on the clayey soils and results indicated recompression indices of about 0.017 and 0.025, and compression indices of 0.16 and 0.17. Overconsolidation ratios were in the order of 6.7 and 5.3. Consolidation test results are presented in Appendix C.

Preliminary anticipated settlement for the proposed shallow foundations is presented in Section 6.2.1. Differential settlement is typically estimated as one-half of total settlement, but may vary depending on foundation type, location, and size. The effects of consolidation settlement on proposed structures are considered to be minimal. Static settlement estimates should be re-evaluated during future design phases once foundation types and loading are known.

4.4 GROUNDWATER

A review of the available groundwater well information from the California Department of Water Resources website (CDWR, 2021) and United States Geological Survey (USGS, 2021a) indicates that there are no wells within a mile radius from the site.

Groundwater was not encountered during our field investigation. Fluctuations of the groundwater level, localized zones of perched water, and an increase in soil moisture should be anticipated during and following the rainy seasons or periods of locally intense rainfall or storm water runoff.

5.0 GEOLOGIC AND SEISMIC HAZARDS

5.1 SEISMIC HAZARDS

5.1.1 Fault Rupture

As mentioned in Section 3.3, the Project alignment is not traversed by known active or potentially active faults. Therefore, the potential for fault rupture within the Project site is considered low. Based on the information available on EQ Zapp (CGS, 2021), the nearest special study zone is the Los Osos fault zone which is approximately 3 miles to the west of the Project site. This Special Studies Zone is shown on Figure 5 in Appendix A.

5.1.2 Liquefaction

The term liquefaction describes a phenomenon in which soils temporarily lose shear strength (liquefy) due to increased pore water pressures induced by strong, cyclic ground motions during an earthquake. Liquefaction is associated primarily with loose to medium dense, saturated, fine- to medium-grained, cohesionless soils. Structures founded on or above potentially liquefiable soils may experience bearing capacity failures due to the temporary loss of foundation support, vertical settlements (both total and differential), and/or undergo lateral spreading. The factors known to influence liquefaction potential include soil type, relative density, grain size, confining pressure, saturation, and the intensity and duration of the seismic ground shaking.

The Project site is located within an area mapped with a low to moderate liquefaction potential (County of San Luis Obispo, 2019a) as shown in Figure 5 in Appendix A. Based on the lack of groundwater in the upper 50 feet, per our investigation, and relatively dense or hard nature of the material encountered, liquefaction is not considered a design issue.

5.1.3 Seismically-Induced Settlement

Seismically-induced settlements consist of dry dynamic settlement (above groundwater) and liquefaction-induced settlement (below groundwater). The dry dynamic settlement occurs primarily within loose to moderately dense sandy soils due to a reduction in volume during and shortly after an earthquake event. Due to the high plasticity and dense/hard nature of the material encountered, the potential for seismically-induced settlement is considered low.

5.1.4 Lateral Spreading

Liquefaction-induced lateral spreading is defined as the finite lateral displacement of ground as a result of pore pressure build-up or liquefaction in shallow underlying soils during an earthquake. Lateral spreading can occur on sloping ground or where nearby steep banks are present. Based on the site configuration (relatively flat terrain with minor slopes) and low liquefaction potential, the potential for lateral spreading is considered to be low.

5.1.5 Seiches and Tsunamis

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. Tsunamis are waves generated in large bodies of water by fault vertical displacement or major ground movement. Considering the Project site elevations, inland location, and absence of enclosed bodies of water near the Project site, seiche and tsunami risks at the site are considered negligible.

5.1.6 Earthquake-Induced Flooding

Earthquake-induced flooding is caused by dam failures or other water-retaining structure failures as a result of seismic shaking. A review of the Dam Inundation Map by the County of San Luis Obispo (CSLO) Department of Planning & Building (CSLO, 2019) indicates that the Project site is not located within an area susceptible to dam inundation. Therefore, the risk related to earthquake-induced flooding is considered to be low for the Project site.

5.2 FLOODING

According to the Federal Emergency Management Agency (FEMA) Flood Insurance Rate Map, Map Number 06079C1069G (FEMA, 2012), the Project site is considered a Zone X (area with minimal flood hazard). The Zone X represents an area that is determined to be outside the 0.2 percent annual chance flood. Therefore, the flooding risk is considered low at the Project site.

5.3 LANDSLIDES

The Project site is located in a relatively flat terrain with the exception of minor slopes (less than 3 feet in height) located adjacent to the railroad tracks. Additionally, the area was not mapped within a landslide zone as shown in Figure 5 (County of San Luis Obispo, 2019a) in Appendix A. Therefore, the risk of landslides at the Project site is considered low.

5.4 EXPANSIVE AND COLLAPSIBLE SOILS

Soil expansion describes the tendency of the soil to expand when wet or contract when dried. Soil collapse indicates the tendency for soil to contract suddenly when loaded and wetted. Based on lab results, in some areas of the site, soils in the upper 5 feet have a moderate potential for expansion. However, as noted in Section 4.3.3, other soil types below a depth of 5 feet may exhibit higher expansion potential. Therefore, the final design should incorporate recommendations to mitigate their effects. Expansion potential should be evaluated based on exact locations, depths, and types of proposed foundations and other at-grade improvements.

5.5 SUBSIDENCE

Subsidence is the sinking of the ground surface caused by the compression of earth materials or the loss of subsurface soil due to underground mining, tunneling, erosion, or pumping/extraction of groundwater. The major causes of subsidence include fluid withdrawal from the ground, decomposing organics, underground mining or tunneling, and placing large fills over compressible earth materials. The effective stress on underlying soils is increased resulting in consolidation and settlement. Subsidence may also be caused by tectonic processes. The Project site is not located in an area of known ground subsidence or within any delineated zones of subsidence due to groundwater pumping or oil extraction (USGS, 2021b). Accordingly, the potential for subsidence to occur at the Project site is low.

6.0 PRELIMINARY GEOTECHNICAL RECOMMENDATIONS

6.1 SEISMIC DESIGN CRITERIA

Seismic design of the proposed structures should be based on the applicable design codes and reviewing agencies for each structure. During this preliminary phase, we have assumed that the final design will be based on a combination of the latest versions of the American Railway Engineering and Maintenance-of-Way Association manual (AREMA, 2019) and the California Building Code (CBC, 2019). Preliminary seismic design parameters for both codes are presented below.

6.1.1 AREMA

A seismic hazard analysis was performed using the USGS Unified Hazard Tool (USGS, 2021c) to evaluate anticipated ground motions at the Project site. Site Class D was used for the Project site. Peak ground accelerations (PGAs) were estimated for upper bound return periods for the three seismic levels recommended in the AREMA manual (AREMA, 2019). These seismic events include Level I (50 to 100-year return period), Level II (200 to 475-year return period), and Level III (1,000 to 2,475-year return period). PGAs for each return period were initially estimated for Site Class B and were then adjusted to Site Class D (assumed Site Class at the Project site). Table presents the results of our preliminary seismic analysis. During future design stages, the return period corresponding to each seismic event should be adjusted using the AREMA risk factors and an acceleration response spectrum (ARS) should be developed for each seismic event in accordance with Chapter 9 of AREMA (2019).

Table 6-1. AREMA Peak Horizontal Ground Accelerations

Seismic Event Level	Return Period (years)	PGA[(1)], g
I	100	0.08
II	475	0.19
III	2,475	0.40

Note:

1. g = unit of gravitational acceleration. USGS (2021c) for Site Class B using Conterminous 2014 dataset (V 4.0.x). Peak ground accelerations are adjusted to the assumed Project site class (Site Class D) from baseline Site Class B data per AREMA (2019).

6.1.2 CBC

Preliminary seismic parameters estimated using the SEA/OSHPD Hazard Tool (SEA, 2021) and in accordance with the 2019 California Building Code (CBC) and ASCE/SEI 7-16 Standard (ASCE, 2017) are presented in Table 6-2. The values presented in Table 6-2 are based on mapped values and are appropriate for conceptual design. Seismic parameters presented in Table 6-2 should be confirmed during the final design phase of the Project.

Table 6-2. Preliminary CBC Seismic Design Parameters

Category	Coefficient
Site Class	D
Latitude	35.2695
Longitude	-120.6535
Mapped (5% damped) spectral response acceleration parameter at short period (0.2 sec), S_S	1.067
Mapped (5% damped) spectral response acceleration parameter at long period (1.0 sec), S_1	0.393
Short period (0.2 sec) site coefficient, F_a	1.073
Long period (1.0 sec) site coefficient, F_v	1.907[(2)]
Spectral response acceleration parameter at short period (0.2 sec), S_{MS}	1.145
Spectral response acceleration parameter at long period (1.0 sec), S_{M1}	0.749
Design (5% damped) spectral response acceleration parameter at short period (0.2 sec), S_{DS}	0.763
Design (5% damped) spectral response acceleration parameter at long period (1.0 sec) S_{D1}	0.500
Peak Ground Acceleration (PGA) (g)	0.473
Site Modified PGA (PGA_M) (g)	0.533
Seismic Design Category [(1)]	D

Notes:

(1) Based on a Risk Category II. Seismic Design Category to be confirmed by structural engineer.

(2) See commentary in ASCE/SEI 7-16, Section 11.4.8 for site-specific ground motion analysis and “Exception note” 2.

6.2 Foundation Types

We understand that the proposed improvements considered for this project include the construction of an administrative office building, train wash facility, layover tracks, and other ancillary improvements. Based on the anticipated structural loading, most of these proposed structures can be supported on spread footings founded on properly prepared subgrade soils. Some of the ancillary improvements may be supported on pole foundations. Recommendations for shallow foundations and pole foundations are provided in this section.

Slabs-on-grade or deep foundations are viable alternatives, but are assumed to not be cost effective based on the current concept. Recommendations for these foundation types are not included in this report, but could be considered during future design phases.

6.2.1 Shallow Foundations

For structures supported on properly compacted subgrade, an allowable bearing capacity of 2,000 pounds per square foot (psf) may be used with a minimum embedment of 24 inches below the lowest adjacent grade, and minimum footing width of 18 inches. This value may be increased by 100 or 1,000 psf per every foot of width or embedment, respectively to a maximum of 3,000 psf.

This value may be increased by one-third when considering loads of short duration, such as those imposed by wind and seismic forces. Overexcavation below the footings is anticipated as described in Section 6.6.2. The allowable bearing capacity has incorporated a factor of safety of 3.0. The total and differential static settlement of foundations designed and constructed per our geotechnical recommendations is expected to be less than one inch and 1/2-inch, respectively The footing dimension and reinforcement should be designed by the Project structural engineer. A coefficient of resistance of 0.40 for lateral sliding resistance may be assumed in the preliminary designs.

6.2.2 Pole Foundations

The following recommendations may be used for the design of pole foundations for catenary poles, canopy pole footings, light poles and signal posts, and similar uninhabited structures. The allowable unit skin friction has incorporated a factor of safety of 2.0 and is intended for a pole foundation with an embedment depth of 12 feet or less. The allowable end bearing capacity has incorporated a factor of safety of 3.0. Skin friction and end bearing can be combined in pole foundation design if the bottom of the shaft is well cleaned and inspected. Settlements of piles generally result from the settlement of the supporting soils and elastic compression of piles. The post-construction settlement due to the axial load is expected to be less than 0.5-inch.

The lateral load design of the poles may be performed using the method for pole design in the California Building Code (2019) Section 1807.3. The recommended design parameters for pole foundations are presented in Table 6-3. Lateral resistances are unfactored, so the structural engineer should apply the applicable factors of safety and/or load factors during design.

Table 6-3. Design Parameters for Pole Foundations

Generalized Soil Type	Depth Below Grade (feet)	Lateral Resistance	Friction Coefficient	Allowable End Bearing (psf)	Allowable Unit Skin Friction (psf) (1) (2)
Clayey Sand with Gravel/ Clay	0-5	300 psf up to maximum 3,000 psf	0.3	2,000	350
Clay	5-10			3,000	

Note:

1. Uplift resistance may be taken as 70 percent of skin friction value. Upper 1.5 pile diameters should be ignored in skin friction capacity.
2. psf = pounds per square foot

6.3 LATERAL EARTH PRESSURES

Retaining walls are not shown on the current conceptual plans for the Project. However, based on discussions with the design team, short retaining walls up to 10 feet may be incorporated into future design phases. Table 6-4 provides a set of equivalent fluid pressure (EFP) values for the preliminary design of earth-retaining structures (cut and fill walls) at the Site. The EFP concept is commonly used in the estimation of the lateral earth pressure which a retaining wall or shoring system will be required to resist. EFP is expressed as the unit weight of a fluid (in pcf) which 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 pounds per cubic foot (pcf) may be used for calculating the actual weight of the soil over a structure.

EFP values were provided 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 cast-in-place diaphragm wall system might experience "At-Rest" conditions; and the resistance at the toe of the shoring might experience "Passive" conditions. Note that lateral earth pressures will be significantly higher for a sloped backfill condition.

Table 6-4. Lateral Earth Pressures

Condition	Equivalent Fluid Pressure (pcf)
	Level Backfill
Active	40
At-Rest	60
Passive	300 to a maximum 3,000 psf
Seismic [(1)]	10, 16

Notes:

1. Seismic pressures provided for AREMA Level II, and apply to active and at-rest conditions, respectively.
 - Values presented in this table do not include a factor of safety.
 - Free-draining soil conditions were assumed.

The above values do not contain an appreciable factor of safety, so the Project structural engineer should use applicable factors of safety and/or load factors during design. The design values indicated above are based upon drained conditions. Proper drainage should be provided behind the walls to prevent buildup of hydrostatic pressure behind the walls, where applicable. Where hydrostatic conditions will be allowed to develop, equivalent fluid pressures should be reduced by 50 percent beneath the water surface and hydrostatic pressures should be added. In addition to the above lateral pressures from retained earth, lateral pressures from other superimposed loads, such as those from adjacent structures or vehicles, should be added per Section 6 of Caltrans *Trenching and Shoring Manual* (Caltrans 2011b). For surcharge loading onto retaining wall structures, loads should be calculated according to AREMA (2019) Chapter 8 Section 20.3.2.

For seismic loading, the pressures presented in Table 6-4 may be used in addition to the static earth pressures for active and at-rest 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. This seismic earth pressure may be assumed to act with a similar load distribution as static pressures.

Backfills for retaining walls, if any, should be compacted to a minimum of 95 percent relative compaction (per ASTM D1557). Retaining walls should be backfilled with non-expansive granular soils, i.e., backfill Types 1 and 2 per Section 5.2.5, Chapter 8 of AREMA (2019). During construction of retaining walls, the backcut should be made in accordance with the requirements of Cal/OSHA Construction Safety Orders. To mitigate the effects of over-stressing the wall, relatively light construction equipment should be used to achieve the compaction requirement behind retaining walls.

6.4 PAVEMENT

Pavement is proposed at the access and service roads, yard, and parking areas. Flexible or rigid pavement sections or a combination of both, depending on use, may be used at the site. Calculations were performed based on an R-value of 18. A traffic index of 5 was assumed to represent lightly loaded parking areas and access roads based on Caltrans Highway Design Manual (2018b). If significant traffic loads are anticipated from maintenance vehicles or other trucks, additional calculations should be performed during future phases of design.

6.4.1 Flexible Pavement

Based on our calculation a minimum of 3 inches of asphalt concrete over 6 inches of aggregate base is recommended. Alternatively, a full depth asphalt section of 7.5 inches may be used. The calculated flexible pavement thickness is in general agreement with the City of San Luis Obispo Standard Plan for Pavement Design (2020). The pavement materials should meet the requirements of the Greenbook (BNi, 2018). Base course should be in accordance with Greenbook Section 202 for crushed aggregate or crushed miscellaneous base. Subgrade in the upper 2 feet below the finish grade or 0.5 feet below the grading plane should be compacted to a minimum relative compaction of 95 percent per ASTM D1557.

6.4.2 Rigid Pavement

Based on the Caltrans Highway Design Manual (2018b), a rigid pavement section consisting of 9 inches of jointed plain concrete over 12 inches of Class 2 aggregate base is recommended for a traffic index of 9 or less. Subgrade in the upper 2 feet below the finish grade or 0.5 feet below the grading plane, whichever is greater, should be compacted to a minimum relative compaction of 95 percent per ASTM D1555.

6.5 INFILTRATION BASIN DESIGN

Although a measured infiltration rate above the generally accepted minimum value of 0.5 inch per hour was measured in the field (see Section 2.2), the use of a dedicated infiltration-only basin is not recommended due to the amount of clay in the upper soils in addition to the existing depth of fill with clayey characteristics, and the potential for expansion in the upper 10 feet. However, best management practice (BMP) may not preclude the use of bioswale-type pretreatment or detention options.

Effective infiltration BMP design requires proper design assumptions and proper device maintenance. The application of each BMP should consider the possible requirements for water pretreatment, device siltation/clogging, consequences of under/over performance, and other considerations. The potential for requiring water pretreatment should be considered, depending on design application. Where infiltration is intended, the soil at the bottom of the proposed BMP should not be compacted, and should be inspected during construction by HDR or our geotechnical representative for consistency with the design recommendations herein.

With time, the bottoms of infiltration systems tend to plug with organics, sediments, and other debris. Long term maintenance will likely be required to remove these deleterious materials to maintain design percolation rates. Restrictions on locations of Infiltration systems include being located at least 10 feet from any existing or proposed foundation system, being located away from slopes, and other considerations based on proposed location of system. Due to the site's proximity to existing and proposed slopes, active rail, and other features, BMP methods should be considered carefully and should be located and designed appropriately. Design plans and proposed infiltration methods should be reviewed by the geotechnical engineer during design. For additional recommendations see the references from Caltrans (2011a).

The potential for underground contamination and the implications of installing a BMP should be considered during design.

6.6 EARTHWORK

6.6.1 Site Preparation

Prior to construction, the Site should be cleared of all existing improvements and debris within the footprint of the proposed improvements plus an offset as judged by the representative of the Project geotechnical engineer. Existing utility and irrigation lines should also be either removed or protected in place, if they interfere with the proposed construction. Cavities resulting from removal of the existing underground structures should be excavated to reach a firm and non-yielding subgrade before being properly backfilled and compacted.

As judged by the Project geotechnical engineer's representative onsite, all deleterious and organic materials exposed at the surface should be stripped and removed until a firm and non-yielding subgrade is reached. Deleterious material may include uncertified, compressible, collapsible, or expansive soils.

6.6.2 Overexcavation

Building Footprints: For building pad areas, in general, pads should be overexcavated to a depth of at least 3 feet below the bottom of footings or to the depth of existing artificial fill, whichever is greater, and the soil should be replaced with engineered fill as defined in Sections 6.6.3 and 6.6.4. The lateral limit of overexcavation and engineered fill should be established at a minimum distance of 5 feet horizontally beyond the building footprint. The extent of removals should be evaluated based upon the soils exposed during grading when direct observation and evaluation of materials are possible. Other local conditions may be encountered which could require additional removals.

Tracks: In track areas, removal and recompaction of approximately 2 feet below the existing grade or 2 feet below the finish subgrade, whichever is greater, should be anticipated. Laterally, these engineered fills should extend a minimum of 2 feet beyond the sub-ballast edges. The extent of removals should be evaluated based upon the soils exposed during grading when direct observation and evaluation of materials are possible. Other local conditions may be encountered that could require additional removals. Ballast and subballast recommendations are provided in Section 6.7.

Pavements and Concrete Flatwork: For pavements and at-grade, exterior concrete flatworks, a minimum of 2 feet engineered fill should be placed below the design finished subgrade. Laterally, these engineered fill should extend a minimum of 2 feet beyond the pavement and flatwork edges.

Local overly wet areas or soft, unstable/pumping subgrade conditions may be encountered during site grading activities. The bottom of the overexcavation may be difficult to compact using conventional methods of fill placement and compaction due to the presence of fine-grained soils with moderate potential for expansion. The contractor should consider the moisture conditions when selecting equipment for earthwork and compaction. During seasonal rains, handling of saturated soils may pose problems in equipment access and cleanup. These conditions could seriously impede grading by causing an unstable subgrade condition. Typical remedial measures include the following:

- Drying: Drying unstable subgrade involves disking or ripping wet subgrade to a depth of approximately 18 to 24 inches and allowing the exposed soil to dry. Multiple passes of the equipment (likely on a daily basis) will be needed because as the surface of the soil dries, a crust forms that reduces further evaporation. Frequent disking will help prevent the formation of a crust and will promote drying. This process could take several days to several weeks depending on the depth of ripping, the number of passes, and the weather condition.

- Removal and Replacement with Crushed Rock and Geotextile Fabric: Unstable subgrade could be overexcavated 18 to 24 inches below planned excavation depth and replaced with crushed rock ranging from ¾ inch to 2 inches in size, underlain by geotextile fabric. The geotextile fabric should consist of a woven geotextile, such as Mirafi 600X or equivalent. The final depth of removal will depend upon the conditions observed in the field once overexcavation begins. The geotextile fabric should be placed in accordance with the manufacturer's recommendations.

- Lime Treatment: Unstable subgrades could be stabilized by mixing the upper 18 to 24 inches of the subgrade with lime. For estimating purposes, 3 to 4 percent for high calcium quick lime may be used. Final application rates should be determined in the field at the time of construction in consultation with the geotechnical engineer.

6.6.3 Engineered Fill

All fill soils should be placed in thin (maximum 8-inch loose thickness, except as noted for oversize materials in Section 6.6.4), horizontal lifts with each lift properly moisture conditioned to about two percent above the optimum moisture content and compacted to a minimum of 90 percent relative compaction per ASTM D1557. Subballast and aggregate base should be compacted to a minimum of 95 percent relative compaction.

6.6.4 Fill Material

Fill and backfill material should be free of organic matter, excessive fines, or unsuitable products of demolition. Granular material with particle size in excess of than 3 inches in diameter should not be placed within 2 feet of the finished grade and oversize material greater than 6 inches in diameter should not be used in structural fill within 8 feet of finished grade. Fill and backfill material should have plasticity index of 15 or less, a liquid limit of 30 or less, expansion index of 30 or less, and a low corrosion potential (classified as non-corrosive by Caltrans, see Section 4.3.3).

Based on atterberg limits and expansion index testing, some of the surficial soils encountered at the boring locations are in general not suitable for use as engineered fill. However, with some mixing or regrading, these soils may be used in the engineered fill provided that they meet the criteria mentioned above.

Structural backfill material at retaining walls should have a sand equivalent of not less than 20. Based on laboratory test result of one sample in the upper 5 feet, the onsite soils do not meet this requirement.

Soils to be placed as fill, whether onsite or import material, should be approved by the Project geotechnical engineer. In general, material such as topsoil, loam, uniform fine sand, silt, and clay should be avoided.

6.6.5 Expansive Soils Mitigation

Testing of samples obtained in the vicinity of the proposed improvements indicated that moderately expansive soils should be expected at this location. Pavement, slab on grade, flatwork, and foundations may be susceptible to damage due to the upper expansive soils at the Project site.

To mitigate the effects of the upper expansive soils, the uppermost 18 inches of soil should be removed and replaced with engineered fill where highly expansive soils exist beneath the concrete flatwork or foundations. If the cost of soil replacement or import fill is prohibitive, it may be cost effective to use lime treatment stabilization in the upper 18 to 24 inches rather than soil removal and replacement as described in Section 6.6.2.

To mitigate impacts of expansive soils, it is critical to minimize seasonal or local fluctuations in subgrade moisture content. This can be achieved by pre-wetting the upper 18 inches of soils prior to pavement, slab on grade, or flatwork construction, and maintaining moisture content about 4 percent over optimum moisture content during and after compaction. All surface runoff should be collected and drained without allowing infiltration to the native soils.

6.7 BALLAST AND SUB-BALLAST

A stable roadbed is critical to provide the foundation upon which ballast, track, and ties are laid and for support of the track structure with limited deflections. The thickness of ballast and subballast sections and dimensions should comply with UPRR Standard 0008A. At a minimum, the upper 24 inches of subgrade should be properly compacted to at least 90 percent relative compaction (ASTM D1557) prior to placing ballast and sub-ballast. Subgrade should be prepared in accordance with Section 6.6 of this report.

The purpose of the sub-ballast is to form a transition zone between the ballast and subgrade to avoid migration of soil into the ballast, and to reduce the stress applied to the subgrade. Sub-ballast should contain no material larger than 3 inches in diameter. Sub-ballast shall be crushed gravel or crushed stone with a minimum 75 percent of the material having two fractured faces. Sub-ballast must meet the quality requirements of ASTM D1241 (e.g. gradation, abrasion loss, liquid limit, etc.) and be approved by the Project geotechnical engineer.

The principal purpose of the ballast section is to support the tracks and provide resistance against lateral, longitudinal and vertical movements of ties and rails (i.e., stability). Additionally, the ballast distributes the applied load on a larger surface area resulting in lower pressures applied to the subgrade, provides immediate drainage for the tracks, facilitates maintenance, and provides a necessary degree of elasticity and resilience. Ideal qualities in ballast materials are hardness and toughness, durability or resistance to abrasion and weathering, freedom from deleterious particles (dirt), workability, compactability, cleanability, and availability. Important ballast properties include shape of the ballast particles, degree of sharpness, angularity, and surface texture or roughness. These factors have been shown to have a significant effect on the stability and compactability of aggregates in general. Ballast and sub-ballast material properties and placement should conform to UPRR standard specifications and drawings.

6.8 Slopes

Although not shown on preliminary drawings, per conversations with the design team, slopes may be considered on the west side of the site to accommodate a trail. For minor slopes (less than 10 feet in height), slopes should be constructed no steeper than 2H:1V (horizontal to vertical) for cut or fill slopes. Slope stability analyses for static and seismic conditions should be performed during final design once slope geometries are known. Proper drainage should be considered for the slopes to prevent soil saturation and buildup of hydrostatic pressures.

In locations where new fill is planned to extend outside of existing slopes, the existing slopes should be completely cleared of all vegetation and bench-cut along their entire height to remove previous erosion channels or slope irregularities. After benching, new fill placement and compaction should be performed in horizontal lifts as described in Section 6.6.4. In order to achieve compaction, the slopes may be overbuilt and cut back to final grade, or they may be surface rolled to provide a compacted finished surface. Runoff should not be permitted to flow over cut or fill slopes in such a way as to cause erosion.

6.9 Trench Backfill

Utility trenches should be backfilled and compacted with fill material in accordance with Section 10.4, Chapter 8 of AREMA (2019) or Sections 306-12 and 306-13 of the Standard Specifications for Public Works Construction, (“Greenbook”). Additionally, the requirements of UPRR Guidelines for Temporary Shoring (UPRR, 2004) applies to all trenches and excavations.

Utility pipes should be placed on properly placed bedding materials extended to a depth recommended in the pipe manufacturer’s specification. The pipe bedding should extend to at least 12 inches over the top of the pipeline. The bedding material may consist of compacted free-draining sand, gravel, or crushed rock. If sand is used, the sand should have a Sand Equivalent value (California Standard Test Method 217) of 30 or greater. The two Sand Equivalent tests performed on subgrade material in this Project indicate that soils are not acceptable for use as pipe bedding (see Appendix C for lab results). Therefore, acceptable pipe bedding may be imported.

Above the bedding zone, trenches can be backfilled with engineered fill. Oversized rock (cobbles and/or boulders) should either be removed from the alignment or pulverized for use in backfill. Gravel larger than ¾ inches in diameter should be mixed with at least 80 percent soil by weight passing the No. 4 sieve. We recommend that the materials used for the bedding zone be placed and compacted with mechanical means. Densification by water jetting should not be allowed.

Backfill for the trenches should be placed in thin lifts, loose lift thickness being compatible with the earthwork equipment but not exceeding 12 inches, moisture-conditioned to up to four (4) percent above optimum moisture content, and mechanically compacted to a minimum 90 percent relative compaction (ASTM D 1557).

6.10 CORROSION MEASURES

A discussion of soil corrosion results is included in Section 4.3.4. The test results included in this report should only be used as a screening process for an indication of soil corrosivity. In general, foundation elements should be designed for a moderately corrosive environment toward buried concrete and ferrous metals. The cement type should be selected in accordance with soil corrosivity results described in Section 4.3.4, and appropriate strength and mix requirements should be selected based on individual structures' design life and structural requirements. For sensitive buried metallic elements, a corrosion engineer should be consulted.

7.0 CONSTRUCTION CONSIDERATIONS

7.1 TEMPORARY EXCAVATIONS AND SHORING

Excavations that are 5 feet or deeper should be laid back or shored in accordance with OSHA requirements before personnel are allowed to enter. A soil Type "C" may be assumed for the onsite soils. For temporary excavations greater than 5 feet deep that cannot be adequately sloped for stability, some form of temporary external support will be required. Selection and design of temporary shoring system should be performed in accordance with OSHA regulations, and completed by a contractor that is familiar with shoring design.

Temporary shoring in the proximity of the railroad track should be designed in accordance with AREMA Chapter 8 Section 28.5 (2019). Shoring should also be designed to resist lateral surcharge from train loading, adjacent vehicular traffic, construction equipment, and existing structures.

7.2 HISTORIC STRUCTURES

We understand that the Project is located at the site of previously demolished railroad structures over 100 years old. Some of these structures, such as part of the former roundhouse, have been determined to be historically significant and will be protected and incorporated into the final Project. Additional debris from the historic structures will likely be found in other areas of the site and may require significant effort to identify and remove as recommended in Section 6.6.1

In addition to the obstruction, the historic railroad operations likely used materials that are currently classified as hazardous materials. Evaluation of potentially hazardous materials is not part of our scope of services and status of any prior testing or remediation at the site has not been provided. We recommend that a hazardous materials remediation or disposal plan be prepared prior to the start of construction activities.

7.3 ADDITIONAL GEOTECHNICAL SERVICES

The proposed construction involves various activities that would require geotechnical observation and testing. These include:

- Plans and specifications review;
- Overexcavation and soil removal and/or exposed excavation bottom;
- Pumping or unstable subgrade;
- Placement of compacted fill;
- Footing excavation; and
- When any unusual subsurface conditions are encountered.

These and other soil-related activities should be observed and tested by a representative of the Project geotechnical engineer.

8.0 LIMITATIONS

This report has been prepared for the use of HDR and the LOSSAN for the proposed Central Coast Layover Facility Project. This report may not be used by others without the written consent of our client and our firm. The conclusions and recommendations presented in this report are based upon the generally accepted principles and practices of geotechnical engineering utilized by other competent engineers at this time and place. No other warranty is either expressed or implied.

Additionally, the conclusions and recommendations presented in this report have been based upon the subsurface conditions encountered at discrete and widely spaced locations and at specific intervals below the ground surface. Soil and groundwater conditions were observed and interpreted at the exploration locations only. This information was used as the basis of analyses and recommendations provided in this report. Conditions may vary between the exploration locations and seasonal fluctuations in the groundwater level may occur due to variations in rainfall and local groundwater management practices. If conditions encountered during construction differ from those described in this report, our recommendations may be subject to modification and such variances should be brought to our attention to evaluate the impact upon the recommendations presented in this report.

9.0 REFERENCES

The following references were used in preparation of this report:

American Concrete Institute, 2019, Building Code Requirements for Structural Concrete (ACI 318-19), June.

American Railway Engineering and Maintenance-of-Way Association (AREMA), 2019, Manual for Rail Engineering.

American Society of Civil Engineers (ASCE), 2017, Minimum Design Loads for Buildings and Other Structures (ASCE/SEI Standard 7-16), Reston, Virginia.

Building News (BNi), 2018, Standard Specifications for Public Works Construction "Greenbook".

California Building Code (CBC), 2019, July 1, effective January 1, 2020.

Caltrans, 2011a, Infiltration Basins Design Guidance, February.

Caltrans, 2011b, Trenching and Shoring Manual, Rev 1, August 2011.

Caltrans, 2018a, Corrosion Guidelines, Version 3.0, March.

Caltrans, 2018b, Highway Design Manual, Sixth Edition.

CDWR. 2021. Water Data Library, from http://wdl.water.ca.gov/waterdatalibrary/ website, observed on February 1.

CGS. 2021. California Earthquake Hazards Zone Application (EQ Zapp, updated April 4), from <https://www.conservation.ca.gov/cgs/geohazards/eq-zapp> website, Observed on February 1.

City of San Luis Obispo, 2020, Standard Specifications & Engineering Standards, August.

County of Los Angeles. 2014. Guidelines for Design, Investigation, and Reporting Low Impact Development Stormwater Infiltration, Administrative Manual, Department of Public Works, (GS200.1), June 30.

County of San Luis Obispo. 2017. Post Construction Requirements Handbook, Version 1.2, Updated March.

County of San Luis Obispo. 2019. Department of Planning & Building, County of San Luis Obispo and Cities Safety Element Update, Natural Hazards Map: Dam Failure Inundation Areas.

County of San Luis Obispo (CSLO). 2021. Department of Planning & Building Geographic Technology & Design, from http://opendata.slocounty.ca.gov/datasets/seismic-hazard-gsa-combining-designation website, Observed on February 1.

FEMA. 2012. Flood Insurance Rate Map, Map Number 06079C1069G, Revised November 16.

Fuller, M., Brown, S., Wills, C. and Short, W. 2015. Geological Gems of California, California Geological Survey under Interagency Agreement C01718011 with California State Parks.

HDR. 2019. Central Coast Layover Facility Expansion, Preliminary Geotechnical Assessment Report, Sites 1 and 2, San Luis Obispo, California, October.

National Association of Corrosion Engineers (NACE), 1984, Corrosion Basics, An Introduction.

Structural Engineers Association of California (SEAOC) and OSHPD, 2021, Seismic Design Maps, https://seismicmaps.org/ website, accessed on May 6.

UPRR, 2004. Guidelines for Temporary Shoring, October 25.

UPRR, 2020. Union Pacific Railroad Engineering Standards, Roadbed Sections for Concrete Tie Track Construction, Standard Drawing 0002, September 21.

USGS, 2020, Quaternary Faults Web Application, Geologic Hazards Science Center – Earthquake Hazards Pogram, from https://usgs.maps.arcgis.com/apps/webappviewer/index.html?id=5a6038b3a1684561a9b0aadf88412fcf Website, Observed on May 10.

USGS. 2021a. USGS Water Data for the Nation, from https://waterdata.usgs.gov/nwis website, Observed on February 1.

USGS, 2021b, Areas of Land Subsidence in California, from https://ca.water.usgs.gov/land_subsidence/california-subsidence-areas.html website, Observed on February 1.

USGS, 2021c, Unified Hazard Tool, from https://earthquake.usgs.gov/hazards/interactive/ Website, Observed on February 1.

Wiegers. M.O. 2010. Geologic map of the San Luis Obispo 7.5' quadrangle, San Luis Obispo County, California: A digital database: California Geological Survey, Preliminary Geologic Maps, scale 1:24,000.

Appendix A - Figures

Appendix B - Geotechnical Boring Logs

Appendix C - Geotechnical Laboratory Testing Results