A Report Prepared for:

RailPros
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GEOTECHNICAL REPORT
RECONSTRUCT A PORTION OF THE SESPE CREEK OVERFLOW RAILROAD BRIDGE
CITY OF FILLMORE, CALIFORNIA

Project No. 2023-010

by

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October 26, 2023

REVISION NO.	DATE	REVISION DESCRIPTION
Draft V1	10/13/2023	Preliminary Draft for Internal Review – 90% Design
Draft V2	10/26/2023	Draft for Agency Review – 90% Design

QUALITY CONTROL REVIEWER

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1 INTRODUCTION

This report presents the results of the geotechnical design services performed by Diaz•Yourman & Associates (DYA) in connection with planning, design, and environmental compliance to reconstruct the Sespe Creek Overflow Railroad Bridge on the Santa Paula Branch Line services ("Project"). The geotechnical services were performed to provide professional services to Ventura County Transportation Commission (VCTC) ("Owner" and “Client”) with DYA as a subconsultant to RailPros. RailPros authorized this work on June 19, 2023, with a written contract.

The Sespe Creek Overflow Railroad Bridge (Bridge) is located at approximately Mile Post 423.44, west of Fillmore, California, as shown on the Vicinity Map.

Figure 1. In early January, heavy rain, stream flow, and debris accumulated during a series of storms and washed out three spans, or approximately 90 feet, of the Bridge. Three spans on the western end of the Bridge were destroyed and require reconstruction to restore pre-disaster design, capacity, and function to resume rail services on the Bridge. Additionally, an earthwork abutment was partially washed out and will be replaced with a concrete abutment and wingwalls.

The approximate layout of the Project improvements is shown on the Site Plan, Figure 2. Project drawings (Railpros, 2023a) are presented in Appendix A.

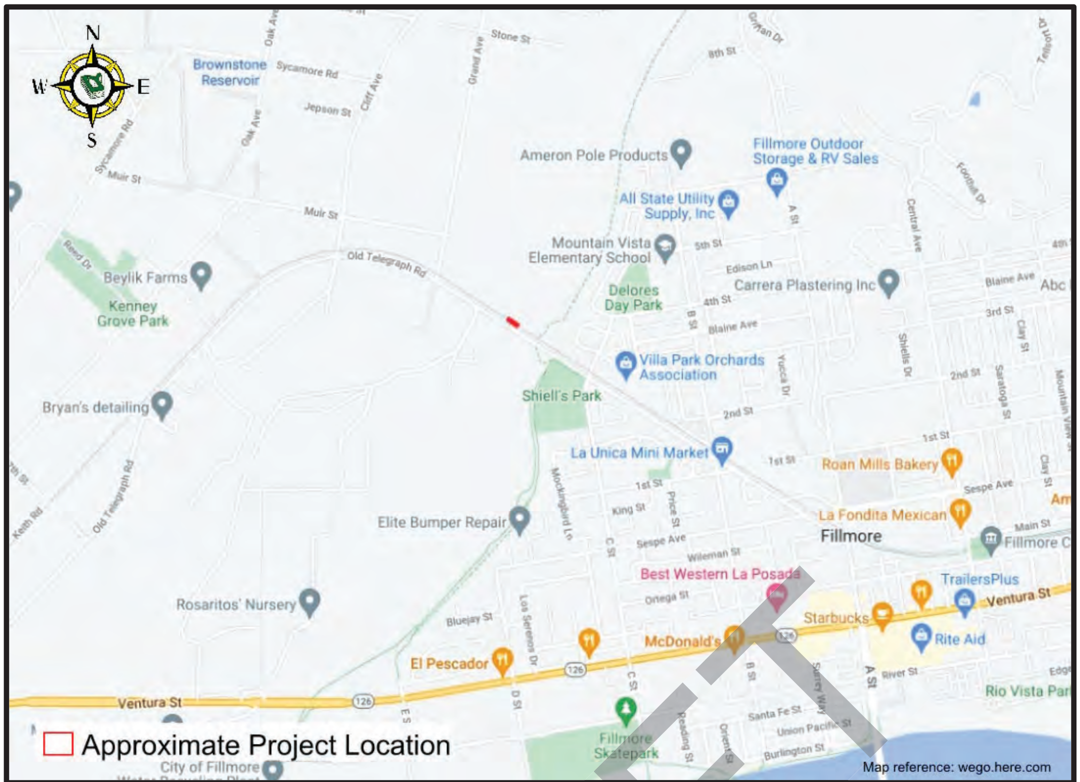


Figure 1 - VICINITY MAP

Path: C:\Users\ashley\diazyourman.com\Projects - Documents\2023\2023-010 VCTC Sespe Creek Rail Bridge\Figures\PRO\InitialPlan.aprx

Figure 2 - SITE PLAN

The purpose of DYA's services was to provide geotechnical input for the design of the Project. The scope of our services consisted of the following tasks:

- Reviewing existing geotechnical and geological data.
- Conducting a limited field exploration.
- Performing limited laboratory tests on selected soil samples.
- Performing engineering analyses to develop conclusions and recommendations regarding the following:
    - Subsurface conditions
    - Geologic and seismic hazards
    - Site preparation and grading
    - Foundation types and deep foundations
    - Estimated total and differential foundation settlement
    - Resistance to lateral loads
    - Lateral earth pressures
    - Soil corrosion potential
- Preparing this report.

Engineering analysis is restricted to the bents and abutment that have currently been observed to have failed. Further analyses for the existing other bridge bents and abutment were not within DYA's scope. Our scope of services also specifically excluded any investigation needed to evaluate the presence or absence of hazardous or toxic materials at the site in the soil, surface water, or groundwater.

2 DATA REVIEW, FIELD EXPLORATION, AND LABORATORY TESTING

The information provided in this report is based on DYA's review of the available regional geologic maps, existing subsurface and groundwater data gathered in the Project vicinity, a limited field exploration, limited laboratory testing, and discussions with Project designer members. Available Caltrans logs of test borings (LOTBs) for the Old Telegraph Road Bridge (Moore and Taber, 1982), which is located adjacent to the failed Bridge, are presented in Appendix B. A list of the documents reviewed is presented in the bibliography (Section 7).

The field exploration, conducted from July 17 through July 26, 2023, consisted of drilling two borings using rotary-wash techniques, each to a depth of approximately 100 feet. The boring locations are shown on Figure 2. One boring (DYB23-02) was drilled on the shoulder of Old Telegraph Road near the location of the washed-out abutment, and the second boring (DYB23-01) was drilled within the Sespe Creek bed near the location of the washed-out bents. As the stream is active in the location of the two washed-out bents, our field exploration was limited to the vicinity of the existing abutment and remaining interior bent. Prior to drilling, the borings were marked and underground service alert (USA) was contacted in order to mark out utility locations. A geophysical survey was also performed prior to drilling to locate any further utilities. Due to the shallow groundwater conditions anticipated at the site, mud-rotary wash drilling techniques were implemented for the field exploration. Because of the difficult access conditions to the channel bottom, a track-mounted, mud rotary wash drill rig was used for the field exploration. In order for the track-mounted, mud rotary drill rig to access the boring location within the creek bed, a pathway was created using a skip loader to move aside cobbles and boulders within the creek bed. Traffic control was provided during drilling and geophysics activities on the roadway. The field exploration implemented standard penetration testing (SPT) to obtain and collect subsurface data and samples for geotechnical engineering properties. Details of the field exploration, including sampling procedures and borings, are presented in Appendix C.

Because of the restrictions to access across the channel bed, drilling deep borings using a drill rig was not possible at the failed bent locations within the three spans on the western end of the Bridge. Therefore, a seismic refraction survey was also performed across the channel bed along the western edge of the Bridge. The location(s) of these seismic refraction survey lines are shown on Figure 2. The purpose of the survey was to develop subsurface velocity profiles of the site and to characterize the subsurface soils at deeper depths (depths deeper than 20 feet) and possibly to estimate the depth to bedrock at the failed bent locations. The refraction survey seismic profiles (Atlas, 2023) are shown in Appendix D.

Soil samples collected from the borings were re-examined in the laboratory to substantiate field classifications. Selected soil samples were tested for moisture content, dry density, grain-size distribution, Atterberg limits, shear strength, and corrosion potential (pH, electrical resistivity, soluble chlorides, and soluble sulfates). The soil samples tested are identified on the boring logs. Laboratory test data are summarized on the boring logs in Appendix C and presented on individual test reports in Appendix E.

3 SITE CONDITIONS

3.1 REGIONAL GEOLOGY

The Project site lies within the east Ventura basin portion of the western Transverse Ranges named for their east-west orientation, roughly perpendicular to most of California's mountain ranges. The east Ventura Basin is generally east-west trending and contains the Santa Clara River into which Sespe Creek drains near Fillmore (

Figure 1 - Regional Geology; Bedrossian and Roffers, 2012). Sespe Creek (including the Project site) contains young wash (river) deposits (map symbol Qw) and is bordered on the west by younger (Qya) and older alluvium (Qoa), and younger alluvial fan (Qyf) deposits. East of Sespe Creek is predominantly Qyf and shale (Tsh) bedrock.

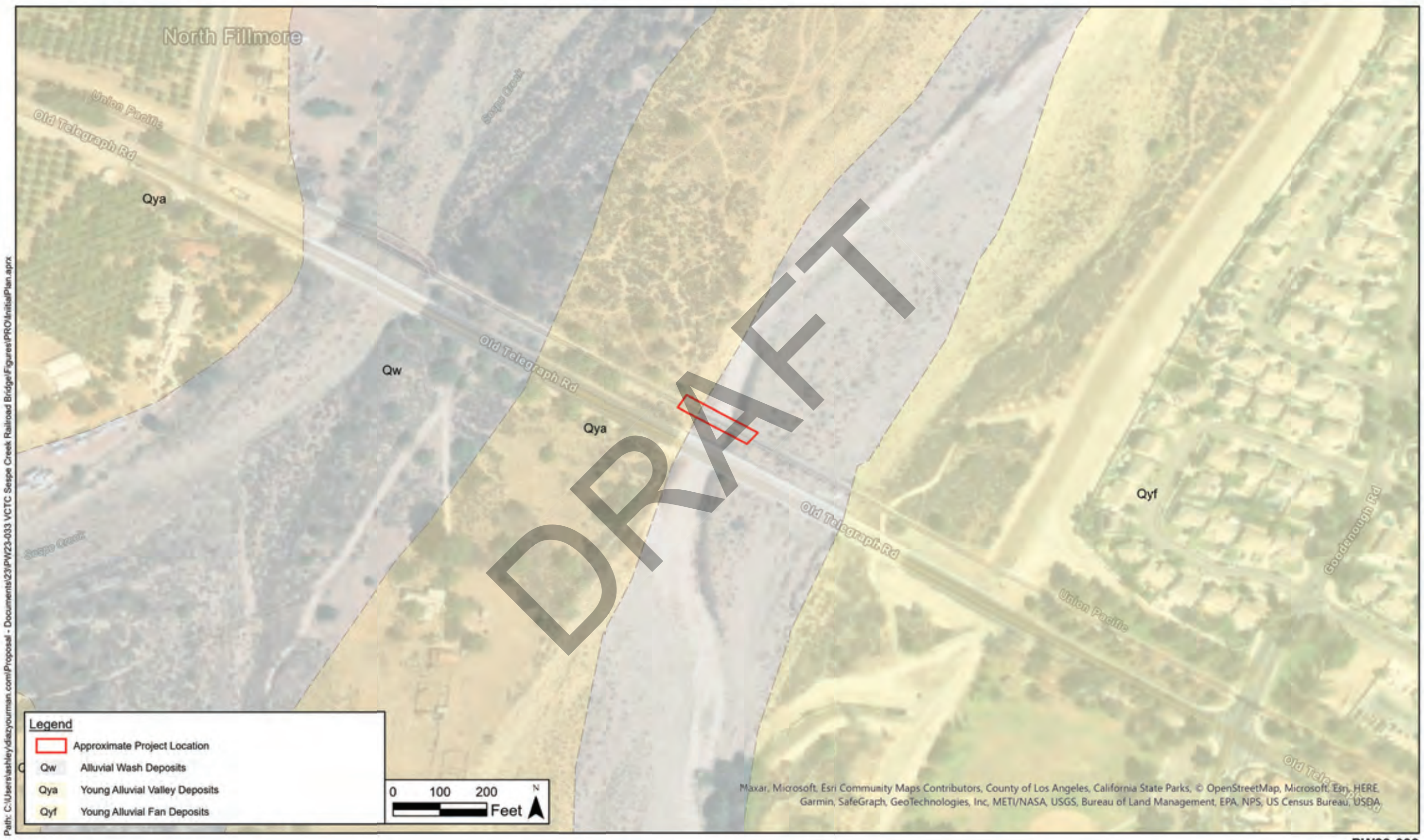
Southern California is a seismically active region with many faults, some of which are capable of producing large-scale earthquakes of approximately 7.0 to 8.0 magnitude (M) on the Richter scale. One such Holocene active fault (Figure 2 - Regional Fault Map; California Geological Survey [CGS], Fault Activity Map website, 2023a) is the San Cayetano Fault that borders the bedrock approximately 5,000 to 10,000 feet east of the Project site and approximately 12,500 feet west of the site. Such earthquakes can trigger severe ground shaking, possible surface fault rupture near the fault, and liquefaction in loose, unconsolidated soils in areas of shallow groundwater.

3.2 LOCAL GEOLOGY

The Project alignment lies within the east Ventura physiographic basin, which is part of the Transverse Ranges geomorphic province. The Santa Clara River-Sespe Creek area of the east Ventura Basin is alluviated lowland that is bound to the north by the Topatopa Mountains and on the south by the Santa Susana Mountains and by South Mountain. The Project site railroad bridge alignment area is mainly mapped as Holocene alluvial wash deposits (Qw) and young alluvium deposits (Qya; Figure 3- Project Site Geology Map).

Qw deposits, beneath the eastern three-quarters of the alignment, are composed of unconsolidated gravel and sand deposits in the active channel deposited from upstream sources in the valley which may contain loose to moderately loose sand and silty sand. Qya deposits, beneath the western one-quarter, are unconsolidated to moderately consolidated boulder, cobble, gravel, sand, and silt deposits. Logs of two test borings (LOTBs B-1 and B-2; Moore & Taber, 1982) just south of the Project alignment indicate that the Qw deposits are 5- to 10-feet thick and consist of coarse gravel, cobbles, and boulders with a medium to coarse sand matrix. LOTB B-1 encountered groundwater at a depth of approximately 40 feet indicating Qya deposits may be susceptible to liquefaction because this is a seismically active region (California Geological Survey, Earthquake Zones of Required Investigation website, 2023b). The nearby San Cayetano reverse fault is believed to be capable of at least a 7.2 magnitude earthquake (Dolan, 2009; Olsen, 2021).

The surface geology units mapped at this site are shown on Figure 3.



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3.3 SURFACE CONDITIONS

At the time of our exploration, two piers of the Bridge had been washed out with a third being pushed out of plumb. The west Bridge abutment was also in the process of failure from erosion. The other intact bridge piers also had a significant buildup of tree debris which may cause significant lateral pressures in the event of another flood. The riverbed was mostly uneven, with numerous small to large boulders. The riverbed had an active stream flowing on the west edge between the west-most pier and the adjoining abutment. The roadway on Old Telegraph Road was in relatively good condition with no noticeable potholes or significant cracks.

3.4 SUBSURFACE CONDITIONS

Based on our limited field exploration, the subsurface soils were significantly difficult to drill through due to the various large-sized boulders encountered and the significant fluid loss experienced. Subsurface soils were primarily sandy gravels, clayey gravel, and silty clayey sands with gravel.

Approximately 20 feet of dense sand and silty sand were present at the abutment location. A five-foot-thick lean clay layer was present at elevation 412 to 407 at the abutment location only. The bottom of the creek bed was estimated to be at elevation 430 feet based on the North American Vertical Datum (NAVD88).

The thicknesses of the different subsurface materials at the abutment location and the channel bottom were idealized along the bridge improvement alignment are presented in Table 1 - IDEALIZED SOIL PROFILE – SESPE CREEK

Note that due to the geological depositional nature of the soils in the creek bed over time, the layers reported in Table 2 may not be present at the same thicknesses at all locations. The site is highly variable with layers of boulders, cobbles, and gravel, and those materials can be encountered at any depth.

Table 1 - IDEALIZED SOIL PROFILE – SESPE CREEK

SOIL LAYER[1,2]	ELEVATION[3] (feet)	DEPTH (feet)	TOTAL UNIT WEIGHT (pcf)	SHEAR STRENGTH		
				Total	Effective	
				S_u (psf[3])	ϕ' (degrees)	c' (psf)
Poorly-Graded Sand with Silt (SP-SM); Silty Sand (SM); ABUTMENT FILL	450 to 430	0 to 20	120	--	34	50
Poorly-Graded Sand with Silt and Gravel (SP-SM); Silty Sand with Gravel (SM); Clayey Sand with Gravel (SC); Poorly-Graded Gravel (GP); CREEK BED	430 to 412[4]	20 to 38	125	--	38	50
Silty Sand with Gravel (SM); Clayey Sand with Gravel (SC); Lean Clay with Sand and Gravel (CL)[5]	412 to 407	38 to 43	125	2,000[5]	38	50
Poorly-Graded Gravel with Silt and Sand (GP-GM); Clayey Sand with Gravel (SC); Silty Sand with Gravel (SM)	407 to 378	43 to 72	125	--	38	50
Clayey Gravel with Sand (GC); Silty, Clayey Gravel with Sand (GC-GM); Silty Sand with Gravel (SM)	378 to 330	72 to 120	125	--	38	50

Note(s):

1. Unified Soil Classification System.
2. Soils are not homogeneous and not in layers. Simplified geotechnical design profile was developed considering the proposed lightly loaded structures and subsurface conditions encountered at the site.
3. Elevation based on NAVD88.
4. Groundwater encountered at an elevation of 423 feet.
5. The 5-foot sandy lean clay layer at elevation 412 to 407 applies to the Abutment 1 location only.

- pcf = pounds per cubic foot.
- The site is highly variable with layers boulders, cobbles, and gravel, and those materials can be encountered at any depth.
- This profile can be used for both the abutments and the bents. See Note 5 for the layer that corresponds to the abutment location only.

3.5 GROUNDWATER LEVEL

Groundwater was encountered during the field exploration in Boring DYB23-01 at 7 feet bgs (elevation 423 feet) and in Boring DYB23-02 at 35 feet bgs (elevation 415 feet). The depth to the historically highest groundwater level near the Project site has been reported to range from 10 to 20 feet (CGS, 2002a). Based on information obtained from the Caltrans LOTBs (Appendix B), the groundwater level was reported at an elevation of 387 feet dating back to 1982 (see Appendix B for details of groundwater elevations encountered). Therefore, the design depth to groundwater was assumed to be at an elevation of 423 feet. Accordingly, design groundwater depth was assumed to be at 7 feet bgs within the creek bed. Note that seasonal variations in water level may occur and that the groundwater can be even closer to ground surface.

4 CONCLUSIONS AND RECOMMENDATIONS

Based on geotechnical considerations, the site is suitable for the proposed Project. The primary geotechnical considerations at the site include the large seismic ground motions, potential liquefaction of loose soils present below the historically highest groundwater levels, scour potential at the abutment locations, and heavy loading of the bridge structure.

The proposed bridge spans at the western end of the Bridge and the abutment can be supported on deep pile foundations. Design recommendations to address the primary geotechnical considerations are presented herein and were developed in accordance with the AASHTO LRFD Bridge Design Specifications (AASHTO, 2017) and the Caltrans Amendments to the AASHTO LRFD Bridge Design Specifications (Caltrans, 2019a).

4.1 SEISMIC/GEOLOGIC HAZARDS

4.1.1 Ground Motion

The site, like most of Southern California, will be subject to strong ground shaking during major earthquakes. The site is outside the Alquist-Priolo Special Study Zone (CGS, 2021) and Landslide Zone (CGS, 2002b). The nearest known active or potentially active faults are summarized in Table 2.

Table 2 - MAJOR FAULT CHARACTERIZATION IN THE PROJECT VICINITY

FAULT[1]	Distance[2] (miles)	SLIP SENSE	DIP (degrees)	DIP (direction)	M_{MAX}
San Cayetano	1.27	Thrust	42	N	7.2
Oak Ridge Connected	2.44	Reverse	53	Unspecified	7.4
Oak Ridge (Onshore)	2.44	Reverse	65	S	7.2
Santa Susana, alt 1	9.91	Reverse	55	N	6.9
Hoser, alt 1	10.39	Reverse	58	S	6.8

Note(s):

1. Based on United States Geological Survey (USGS) online Seismic Hazard Maps (USGS, 2023a).
2. Distance to nearest portion of the project.

- M_{MAX} = maximum earthquake magnitude.
- N = North, S = South

Design earthquake magnitudes ranged from 6.8 to 7.4 for the return periods (USGS, 2023a).

Seismic hazard analyses for the bridge structure consisted of development of acceleration response spectra (ARS). The American Railway Engineering and Maintenance-of-Way Association (AREMA) guidelines (AREMA, 2021) were used for the evaluation of the rail bridge structure in accordance with the SCRRA Design Criteria Manual (2021a).

Seismic hazard analyses were performed using a probabilistic approach in accordance with Chapter 9 of the AREMA manual (2021). The AREMA manual specified three ground-motion levels, which correspond to three performance criteria: serviceability, ultimate, and survivability for seismic design. Probabilistic seismic hazards were evaluated for the Project using the USGS Unified Hazards tool (USGS, 2023b). The return periods and the corresponding peak ground acceleration (PGA) values corresponding to each of the three design ground motion levels are summarized in Table 2. The horizontal acceleration coefficients and return period relationship for the proposed site are summarized in Table 3.

Table 3 - SUMMARY OF AREMA PEAK GROUND ACCELERATIONS

AREMA SEISMIC GROUND MOTION LEVEL	PERFORMANCE CRITERIA	RETURN PERIODS (years)	PEAK GROUND ACCELERATION (PGA, g)
1	Serviceability	95	0.19
2	Ultimate	475	0.44
3	Survivability	2,475	0.82
Note(s) • Values presented in table are based on return periods stated in the SCRRA Design Criteria Manual (SCRRA, 2021a and AREMA, 2021).			

Table 4 - AREMA SEISMIC RESPONSE COEFFICIENTS

PERIOD (seconds)	AREMA SEISMIC RESPONSE COEFFICIENT (C_m)[1,2,3]		
	95-Year Return Period[4]	475-Year Return Period[5]	2,475-Year Return Period[6]
	C_m (g)	C_m (g)	C_m (g)
0.01	0.1932	0.4390	0.8190
0.05	0.2938	0.6106	1.2178
0.10	0.4313	0.9521	1.9670
0.20	0.4313	0.9521	1.9670
0.30	0.4313	0.9521	1.9670
0.40	0.4169	0.9521	1.9670
0.50	0.3335	0.8549	1.7225
0.60	0.2780	0.7124	1.4354
0.70	0.2382	0.6107	1.2304
0.80	0.2085	0.5343	1.0766
0.90	0.1853	0.4750	0.9569
1.00	0.1668	0.4275	0.8613
1.10	0.1516	0.3886	0.7830
1.20	0.1390	0.3562	0.7177
1.30	0.1283	0.3288	0.6625
1.40	0.1191	0.3053	0.6152
1.50	0.1112	0.2850	0.5742
2.00	0.0834	0.2137	0.4306
2.50	0.0667	0.1710	0.3445
3.00	0.0556	0.1425	0.2871
3.50	0.0476	0.1221	0.2461
4.00	0.0417	0.1069	0.2153

Note(s):

1. Seismic response spectra determined in accordance with AREMA, 2021.
2. Seismic response coefficient for the m^{th} mode, C_m, per AREMA (2021), Chapter 9, Paragraph 1.4.4.3.
3. Low period reduced response may be calculated in accordance with AREMA (2021), Chapter 9, Paragraph 1.4.4.4; seismic response coefficient above does not include this adjustment.
4. Level 1 Seismic Ground Motion (AREMA, 2021) corresponding to Earthquake return period of 95 years; Site Class D.
5. Level 2 Seismic Ground Motion (AREMA, 2021) corresponding to Earthquake return period of 475 years; Site Class D.
6. Level 3 Seismic Ground Motion (AREMA, 2021) corresponding to Earthquake return period of 2,475 years; Site Class D.

Ground motion and acceleration response spectra (ARS) were also evaluated using the USGS Unified Hazard Tool (2023b) and Caltrans Seismic Design Criteria (2019b), respectively. The Caltrans procedure considers probabilistic response spectra based on a 5% probability of exceedance in 50 years (975-year return period). Based on the results obtained from the Caltrans ARS online tool (2023) and the USGS Unified Hazard Tool (2023b), the peak ground acceleration (PGA) and earthquake modal magnitude, respectively, for the Project location are presented in Table 3. Caltrans design ARS for the Project are presented in Table 6

Table 5 - SUMMARY OF CALTRANS SEISMIC DESIGN PARAMETERS

Location	Magnitude[1]	PGA[2]
34.406311º, -118.931937º	7.15	0.72

Note(s):

1. Based on United States Geological Survey (USGS) Unified Hazard Tool (USGS, 2023b). Magnitude is based on the maximum value of the mean and modal magnitude values.
2. Based on Caltrans ARS Online Tool V3 (Caltrans, 2023).

Table 6 - CALTRANS ACCELERATION RESPONSE SPECTRUM

PERIOD (seconds)	SPECTRAL ACCELERATION (g)
0	0.72
0.1	1.33
0.2	1.73
0.3	1.79
0.5	1.53
0.75	1.33
1	1.14
2	0.56
3	0.36
4	0.26
5	0.20

Note(s):

- Based on United States Geological Survey (USGS) Unified Hazard Tool (USGS, 2023b). Magnitude is based on the maximum value of the mean and modal magnitude values.
- Based on Caltrans ARS Online Tool V3 (Caltrans, 2023).

4.1.2 Liquefaction Potential

Depth to groundwater was assumed to be at elevation 423 feet at the site. Due to the presence of dense to very dense cohesionless soils at the two bridge sites below the design groundwater level, the potential for liquefaction is considered to be low. Therefore, seismic-induced settlements at the site are anticipated to be minimal. Since the site is not located near a free-face, we judge that potential for lateral spreading is low.

4.2 EARTHWORK

Earthwork is anticipated to be required for the bridge bents and abutment. Deep excavations may be required with shoring adjacent to the roadway or other structures for the construction of the concrete abutment and wingwalls.

4.2.1 Site Preparation and Grading

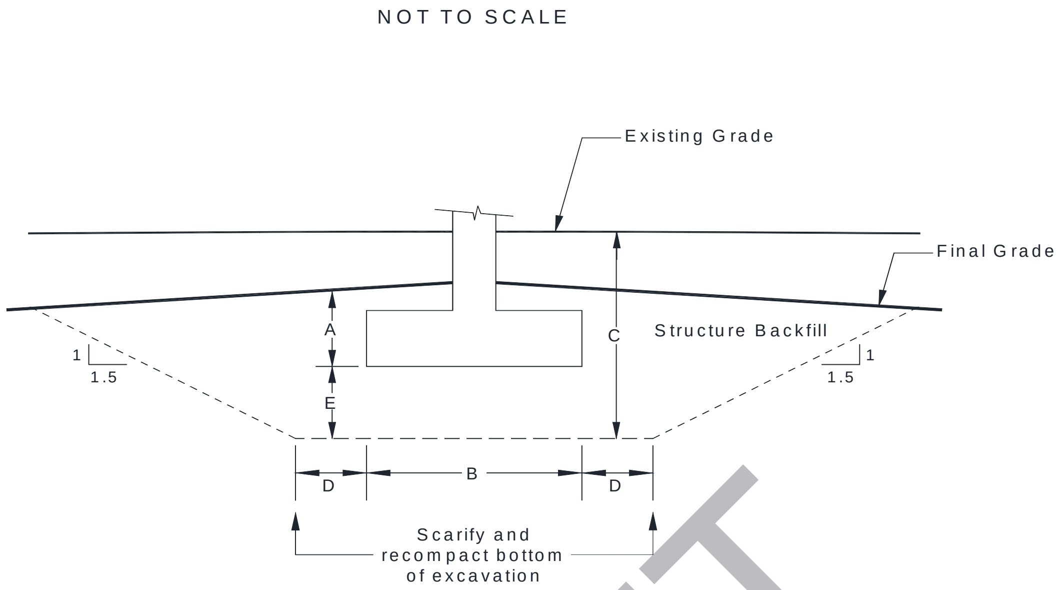
Prior to the start of construction, the following should be performed:

- All utilities should be located in the field and rerouted, removed, abandoned, or protected where necessary.
- Areas to be graded should be stripped of vegetation and debris, and the material removed from the site.
- Pavement should be separated for recycling.

The upper soil should be excavated and replaced with compacted fill as shown on Figure 4. For the bottom of the excavation, the following should be performed:

- Scarified to a depth of 8 inches.
- Moisture-conditioned to at least 2% above optimum moisture content.
- Compacted to at least 95% relative compaction.[1]

[1] Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by ASTM International (ASTM) D1557 test method. Optimum moisture content is the moisture content corresponding to the maximum dry density, as determined by the ASTM D1557 test method.



LOCATION	MINIMUM DIMENSIONS (feet)
A. Footing Embedment Below Final Grade	2
B. Footing Width	2
C. Excavation Below Existing Grade	2
D. Excavation Beyond Footing	5
E. Compacted Soil/Fill Below Footing	2
PRESSURE (psf)	
Static (net) Allowable Bearing Capacity (FS≥3)	5,000
Maximum Static Bearing Capacity (FS≥3)[1]	5,000
Maximum Transient Bearing Capacity ((FS≥2)	7,500

Note(s):

1. The static bearing capacity can be increased by 33% to calculate the transient bearing capacity for transient loading conditions.

- FS = factor of safety.

Figure 4 - GRADING/FOUNDATION DETAILS (LIGHTLY LOADED SHALLOW FOOTINGS)

Where the soils at the bottom of the excavation preclude compaction, they should be excavated to a depth sufficient to achieve a firm and unyielding surface at the planned bottom of excavation or the base of fill. Generally, an overexcavation depth of 1 to 2 feet is sufficient. Using geogrids and/or easily compactable material such as crushed rock can reduce the depth of excavation. The geogrids and/or geotextile should satisfy the requirements of Standard Specifications for Public Works Construction ([Greenbook]; Building News, 2018, Table 213-5.2 (D) Biaxial S1.).

Fill and backfill should be compacted by:

- Placing in loose layers less than 8 inches thick.
- Moisture-conditioning to at least 2% above optimum moisture content.
- Compacting to at least 95% relative compaction.

The compacted subgrade soils should be firm, hard, and unyielding.

Concrete flatwork (i.e., hardscape, curbs, and gutters) should be underlain by a minimum of 12 inches of soil compacted to at least 95% relative compaction and at least 2% above optimum moisture content.

Materials for structure backfill should meet the criteria per SCRRA (2021b) Standard Spec 31.20.00. Recommendations provided in Caltrans specifications (Caltrans, 2018)/Greenbook (Building News, 2018)) can be used for import fill material criteria.

Generally, the upper soils encountered in the borings are not expected to meet the criteria for structure backfill per SCRRA Standard Spec 31.20.00 (SCRRA, 2021b).

Site grading may be accomplished with conventional heavy-duty construction equipment. The fill should be compacted using soil compactors as recommended by the Caterpillar Performance Handbook (2018), or equivalent. However, to avoid overstressing retaining walls when placing backfill adjacent to the retaining walls, backfill should be compacted using lightweight compaction equipment or the walls should be braced.

4.3 FOUNDATION DESIGN

4.3.1 Deep Foundations

We judge that the proposed abutments and bents for the structure replacement can be supported on pile foundations. Drilled (cast-in-drilled-hole [CIDH]) piles were considered for the design. Because of potential driving difficulties/refusals in very dense sands, potential pile-driving-induced vibration, and proximity of rail tracks, driven piles may not be feasible at this Project site. Therefore, CIDH piles were selected by the designer for foundation support for the design.

Construction of CIDH concrete piles should address potential caving/sloughing/heaving of granular soils. Based on the subsurface conditions at the site, the CIDH pile tip elevations are anticipated to be below the design groundwater elevation; therefore, wet construction methods are anticipated for CIDH pile construction. It is likely that CIDH pile construction would require a temporary casing or wet drilling method depending on the anticipated groundwater table at the time of construction. Details of CIDH pile construction considerations are discussed in Section 4.6 of this report

For the design, 6-foot-diameter and CIDH piles were selected by the designer for bent and abutment piles, respectively. Pile axial resistances of 6-foot CIDH concrete piles for the abutment and bents were evaluated using SHAFT (Ensoft, 2017) for the Strength Limit and Extreme Limit cases. The CIDH pile axial compression resistance was based on skin friction and neglecting end bearing resistance. An LRFD Strength and Extreme Limit State resistance factor of 0.7 and 1.0, respectively, were considered for skin friction resistance. Based on the AASHTO LRFD Bridge Design Specifications Section 10.8.3.6.3, for a pile group in sand the individual nominal resistance of each pile should be multiplied by an efficiency factor, η, based on pile center-to-center spacing. Based on the bent layouts, the proposed pile center-to-center spacing of the two 6-foot diameter CIDH piles placed in a single row is 18 feet, or 3 diameters (3D). Based on the abutment layout, the proposed pile center-to-center spacing of the four 6-foot diameter piles, placed in a 2 x 2 group, is 18 feet, or 3D. Therefore, pile group reduction factors of 1.0 and 0.8 were applied in the analyses for the bent and abutment pile axial resistances, respectively. Although our borings were performed only to 100 feet deep bgs, our vertical pile capacity analysis on the creek bed (bent) and abutments were performed to a depth of 120 feet by extrapolating the available soil strength parameters from 100 feet to 120 feet.

Based on discussions with the design team, the pile lateral capacity will be performed by the structural engineering team. The structural engineer will provide the recommended pile lengths from their lateral capacity analyses.

Scour is a design concern because the bridge is located within an active streambed. The calculated long-term, local, and total scour depth and the total scour elevation can be found in Table 7 and Table 8, respectively, in the Hydraulics Report for the Sespe Creel Overflow Channel Railroad Bridge prepared by GHD (GHD, 2023). Bottom-of-scour elevations were provided by Railpros (2023b) in accordance with Section 3.7.5 of the Caltrans Amendments to the AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications (Caltrans, 2019a). The proposed bent and abutment piles should be designed for the local scour, while protecting against potential long-term degradation. Section 7 of the GHD (2023) report provides recommended scour protection countermeasures. Note that the pile cut-off elevations provided in Table 7 and Table

9 were provided by Railpros at a later date, and thus supersede the pile cut-off elevations provided in Appendix A.

The Project structural designer provided the foundation design data, factored design loads and bottom-of-scour elevations for the proposed bent and abutment piles. The foundation design data and bottom-of-scour elevations are presented in Table 7. The factored design loads are presented in Table 8, below. The foundation design recommendations table and pile data table are presented in Table 9 and Table 10, respectively. Settlement of the piles due to Service Limit loading was estimated to be less than 1 inch.

Table 7 - FOUNDATION DESIGN DATA SHEET

SUPPORT NO.	PILE TYPE	CUT-OFF ELEVATION[1] (feet)	BOTTOM-OF-SCOUR ELEVATION (FEET)			PERMISSIBLE SETTLEMENT UNDER SERVICE LOAD (inches)	NUMBER OF PILES PER SUPPORT
			STRENGTH LIMIT STATE	SERVICE LIMIT STATE	EXTREME LIMIT STATE		
Abutment 1	6-foot CIDH	420.75	423.7	411.9	435.4	1”	4
Bent 2	6-foot CIDH	425.00	412.2	406.3	422.1	1”	2
Bent 3	6-foot CIDH	429.00	414.5	406.6	422.4	1”	2

Note:

1. Provided by the structural design team (Railpros, 2023b).

Table 8 - FOUNDATION FACTORED DESIGN LOADS

SUPPORT NO.	SERVICE LIMIT STATE TOTAL LOAD PER PILE (KIPS)	STRENGTH/CONSTRUCTION LIMIT STATE (kips)		EXTREME EVENT LIMIT STATE (kips)	
		COMPRESSION MAX. PER PILE	TENSION MAX. PER PILE	COMPRESSION MAX. PER PILE	TENSION MAX. PER PILE
Abutment 1	887	1,426	0	716	0
Bent 2	550	939	0	778	304
Bent 3	550	939	0	778	304

Note:

- The pile tip elevations should also be checked for lateral loading.

Table 9 - DEEP FOUNDATION DESIGN RECOMMENDATIONS

SUPPORT LOCATION	PILE TYPE	CUT-OFF ELEVATION (feet)	SERVICE-LIMIT STATE LOAD PER PILE (kips)	TOTAL PERMISSIBLE SETTLEMENT SUPPORT SETTLEMENT (inches)	REQUIRED FACTORED NOMINAL RESISTANCE PER PILE (kips) STRENGTH LIMIT COMP. (ϕqs = 0.7)	STRENGTH LIMIT TENSION (ϕqs = 0.7)	EXTREME EVENT COMP. (ϕqs = 1.0)	EXTREME EVENT TENSION (ϕqs = 1.0)	DESIGN TIP ELEVATIONS (feet)	SPECIFIED TIP ELEVATIONS (feet)
Abutment 1	72" CIDH	420.75	887	1	1,426	--	716	--	322.75 (a-I) 385.75 (a-II) 372.75 (c) --[1] (d)	322.75
Bent 2	72" CIDH	425.00	550	1	939	--	778	304	353.0 (a-I) 381.0 (a-II) 397.0 (b-II) 371.0 (c) --[1] (d)	353.00
Bent 3	72" CIDH	429.00	550	1	939	--	778	304	355.0 (a-I) 381.0 (a-II) 397.0 (b-II) 371.0 (c) --[1] (d)	353.00

Notes:

1. Design tip elevations for Lateral Load (d) for Bents are not required per discussion with structural engineer. Based on the lateral loads provided, it is assumed that pile tip elevation per lateral load will not control.

- Design tip elevations are controlled by: (a-I) Compression (Strength Limit), (b-I) Tension (Strength Limit), (a-II) Compression (Extreme Event), (b-II) Tension (Extreme Event), (c) Settlement, (d) Lateral Load.
- The specified tip elevation shall not be raised above the lowest design tip elevation.
- Unsuitable soil layers (i.e., scourable) that do not contribute to the design nominal resistance exist for Abutment 1 and Bents 2 and 3. Bottom-of-scour elevation varies. See Table 7 for bottom-of-scour elevations

Table 10 - PILE DATA TABLE

LOCATION	PILE TYPE	NOMINAL RESISTANCE (kips)		DESIGN TIP ELEVATION (feet)	SPECIFIED TIP ELEVATION (feet)
		COMPRESSION	TENSION		
Abutment 1	6-foot CIDH	2,037	--	322.27 (a) 372.75 (c) -- (d)	322.75
Bent 2	6-foot CIDH	1,174	304	353.0 (a) 397.0 (b) 371.0 (c) -- (d)	353.0
Bent 3	6-foot CIDH	1,174	304	355.0 (a) 397.0 (b) 371.0 (c) -- (d)	355.0

Notes :

- Design tip elevations for abutment and bents are controlled by: (a) Compression, (b) Tension, (c) Settlement, (d) Lateral Load.
- The specified tip elevation should not be raised above the lowest tip elevation.
- Design tip elevation for Lateral Load to be performed by the structural engineer.

4.4 RESISTANCE TO LATERAL LOADS AND LATERAL EARTH PRESSURES

4.4.1 Temporary Shoring

Shoring may be required if excavations for the wingwall are performed adjacent to existing railroad tracks and/or the roadway to avoid undermining the railroad tracks. The contractor should also be prepared to adjust the construction methods based on actual field conditions.

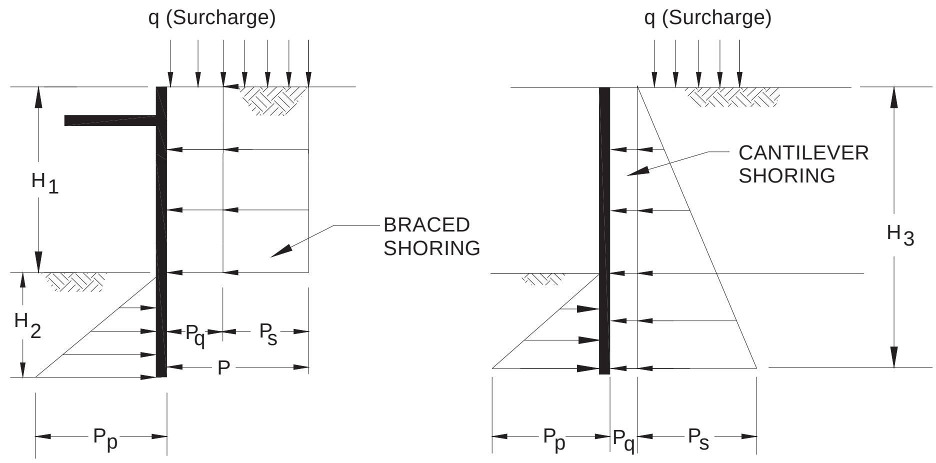
The shoring design is the responsibility of the contractor and should be designed by a registered engineer retained by the contractor. Design of the shoring system will require careful consideration of the existing adjacent improvements, utilities, and foundation systems located close to shored excavations. Shoring design should consider the possible related effects on the surrounding buildings and utilities, deflections of the shoring elements, possible effects of nearby foundation loads on the shoring, and settlements of the retained soil.

The temporary shoring design should incorporate the expected construction procedures, sequence, and loads. In particular, the stockpiling of excavated materials should be considered in design, as well as steel plates for cross traffic and the presence of heavy construction equipment or spoil piles next to the excavations.

Shoring is usually designed as either cantilever (unbraced) or braced. Cantilevered shoring is commonly constructed by either using soldier piles with lagging placed between piles or using sheet piles. If soldier piles and lagging are used, continuous lagging will be required. Difficulty in installing the lagging due to caving cohesionless soils should be anticipated. SCRRA restrictions on the use of shoring based on distance from the operating railroad tracks should be followed (SCRRA, 2021a).

For cantilever shoring, a deflection of 0.5% of the shored height (H1) is necessary to develop active earth pressure (Figure 5 for definition of H1). For braced shoring, the deflection should be less than that for cantilever shoring. We recommend that the design of temporary shoring be performed using shoring pressures equal to or greater than those shown on Figure 5 and Figure 6 and passive resistance equal to or less than that shown on Figure 5. The allowable passive soil pressure outlined on Figure 5 assumes undisturbed existing soils. The upper 1 foot of passive resistance should be neglected.

In using Figure 5, lateral pressures due to rail surcharge on temporary shoring located parallel to the rail tracks can be conservatively estimated assuming lateral coefficients of 0.3 and 0.5 for cantilever and restricted conditions, respectively, and a uniform rail surcharge load (AREMA, 2021).



	BRACED SHORING	CANTILEVER SHORING
$P_p = 390\ H_2 \leq 4{,}000$ psf	$P = P_s + P_q = 24H_1 + 0.5q$	$P = P_s + P_q = 37H_3 + 0.3q$
	(300 psf minimum)	(300 psf minimum)

Note(s):

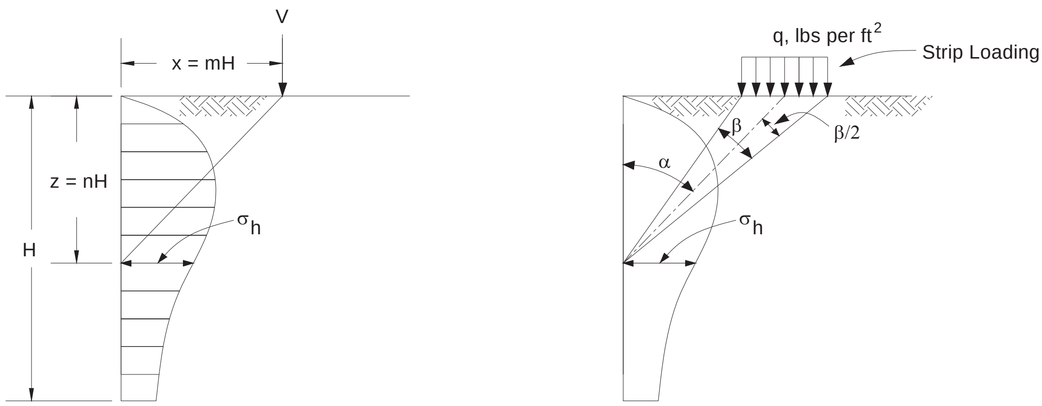
- All values of height (H) in feet and pressure (P) and surcharge (q) in psf.
- Where vehicular traffic from freeway is applicable, assume no less than a 240 psf uniform horizontal pressure.
- Where train load is applicable, use q = live load (from train) + impact load (if considered due to train derailment) per AREMA, Chapter 8, Section 2.2.3.
- Earth pressures assumed no hydrostatic pressures. Hydrostatic pressures, if anticipated, should be added to lateral earth pressures.

Figure 5 - LATERAL EARTH PRESSURES FOR TEMPORARY STRUCTURES

The shoring system and adjacent buildings should be monitored using “real time” instrumentation and optical surveys to check for the lateral and vertical movements until the permanent structure is in place. If large deflections (greater than 0.25% of the shoring height) are noted, the bracing system should be checked and strengthened as needed. If tension cracks appear in the ground surface adjacent to the shoring, the cracks should be monitored and sealed to prevent infiltration of water, and the significance of the cracks should be evaluated immediately.

The type of shoring will depend on the contractor’s means and methods. The excavations should only remain open for very short periods of time.

In addition, the contractor should strictly adhere to any requirements of SCRRA (2021a) and applicable federal and state health and safety regulations such as those of the Occupational Safety and Health Administration (OSHA, 2020). In accordance with OSHA regulations, the near-surface on-site soils are classified as Type C.



POINT LOAD	STRIP LOAD
For $m > 0.4$, $\sigma'_h = \dfrac{1.77V}{H^2}\dfrac{m^2n^2}{\left(m^2+n^2\right)^3}$ For $m \le 0.4$, $\sigma'_h = \dfrac{0.28V}{H^2}\dfrac{n^2}{\left(0.16+n^2\right)^3}$	$\sigma'_h = \dfrac{2q}{\pi}\left(\beta - \sin\beta\cos2\alpha\right)$

Note(s):

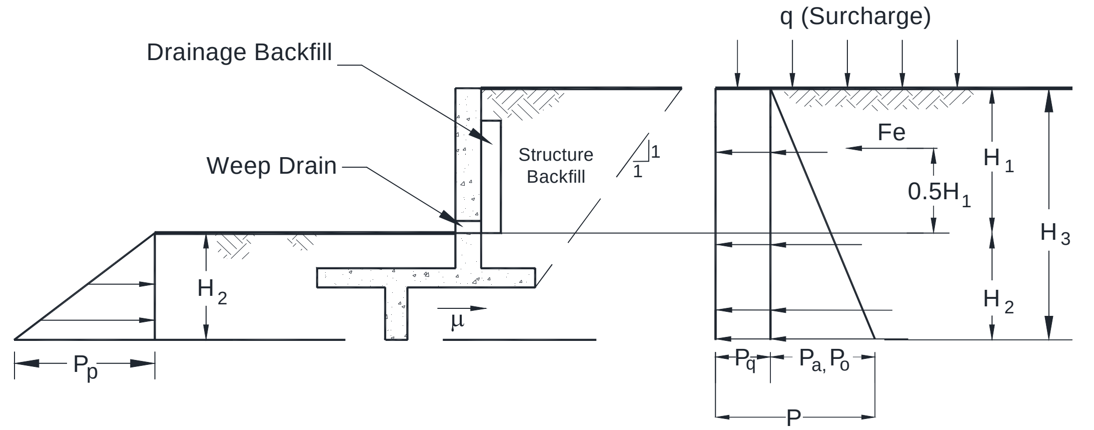
- σ'_h = horizontal stress (psf).
- V = vertical point load (pounds).
- q = vertical strip load (psf).
- H = height of wall (feet).
- α, β = angles (radians).
- m, n = dimensionless coefficients.

Figure 6 - SURCHARGE LATERAL PRESSURE DISTRIBUTION AGAINST A WALL

4.4.2 Permanent Structures

Lateral loads may be calculated per AREMA Chapter 8, Part 5, using trial wedge analysis with a soil friction angle of 32 degrees and soil density of 120 pcf. Lateral loads may also be calculated using Figure 7. Earth pressure coefficient calculations are provided in Appendix G.

The lateral resistance may be calculated using the following: 50% of passive resistance plus 50% of base friction, 100% passive resistance only, or 100% of the base friction only. Lateral loads can be resisted by an allowable passive soil pressure and base friction, as outlined on Figure 7for compacted fill, applied against below-grade walls and foundation elements.



$P_p = 390\ H_2 \leq 4{,}000$ psf	**CANTILEVER WALLS**	**RESTRAINED WALLS**
μ = 0.6 (nominal)	$P = P_a + P_q = 37\ H_3 + 0.3q$	$P = P_o + P_q = 56\ H_3 + 0.5q$
	INCREMENTAL SEISMIC FORCE	
	Serviceability[1]	$Fe = 4\ H_1^2$
	Ultimate[1]	$Fe = 9\ H_1^2$
	Survivability[1]	$Fe = 21\ H_1^2$
	Caltrans 975-year ARP[2]	$Fe = 17\ H_1^2$

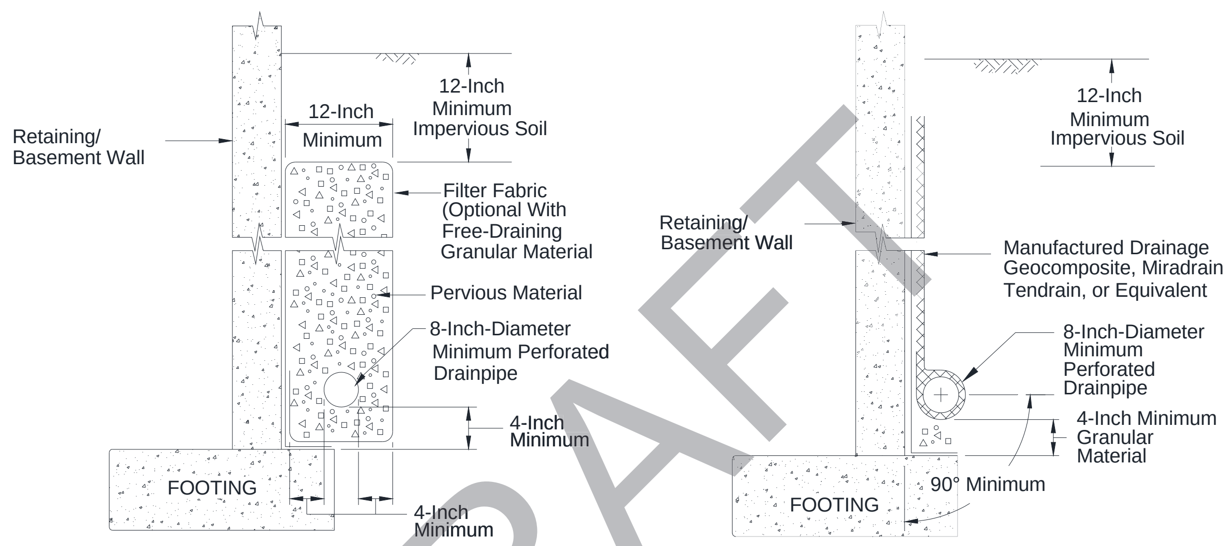
Note(s):

1. Per AREMA (2021) seismic design criteria, PGA_M = 0.193g, 0.439g, and 0.819g were used, respectively, for the Serviceability, Ultimate, and Survivability cases.
2. Per Caltrans ARS Online Tool V2 (Caltrans, 2023), PGA_M = 0.721g

- Lateral earth pressures were calculated using assumed abutment fill properties, including a unit weight of 120 pcf and a friction angle of 32 degrees.
- One-half of the PGA_M was used to calculate Fe.
- All values height (H) in feet, pressure (P) and surcharge (q) in pounds per square foot (psf), and force (F) in pounds.
- Where vehicular traffic from freeway is applicable, assume no less than a 240 psf uniform horizontal pressure.
- Where train load is applicable, use q = live load (from train) + impact load (if considered due to train derailment) per AREMA, Chapter 8, Section 2.2.3.
- P_p, P_a, and P_o are the passive, active, and at-rest earth pressures, respectively; Fe is the incremental seismic force.
- P_q is the incremental surcharge pressure; μ is the allowable friction coefficient applied to dead normal (buoyant) loads. Fe is in addition to the active and at-rest pressures. Below groundwater, in areas of potential pipeline rupture or areas of potential surface water infiltration, active and at-rest pressure should be reduced by 50% and hydrostatic pressure should be added to active and at-rest pressures. P_p should be reduced by 50% below the groundwater.
- For 2H:1V slopes above the wall, increase the active and at-rest pressures by 50%; for 1.5H:1V slope, increase the active and at-rest pressures by 100%.
- Neglect the upper 1 foot for passive pressure unless the surface is contained by a pavement or slab.

Figure 7 - LATERAL EARTH PRESSURES (PERMANENT STRUCTURES)

Retaining walls should be designed to resist lateral earth pressures with equivalent fluid pressures as illustrated on Figure 7. Lateral earth pressures are presented for walls free to rotate and restrained walls. At-rest earth pressures (restrained walls) should be used where the top of the wall is not expected to move laterally more than 0.001 H_1 (see Figure 7). The lateral earth pressures on Figure 7 are based on the structure backfill material noted in Section 4.2.1. The retaining walls should include a drain or be designed for hydrostatic pressure. See Figure 8 for typical sections of wall drains. The train surcharge pressures should be added to the lateral earth pressures on Figure 7 for the retaining wall for the total lateral pressure following the procedure discussed in Section 4.4.1. The seismic earth pressures provided on Figure 7 are based on the PGA_M based on ICC 2019 criteria discussed in Section 4.1.



MATERIAL	CALTRANS SPECIFICATIONS	GREENBOOK SPECIFICATIONS
Free-Draining Granular Material	68-2.02 (Class 2)	300-3.5.2
Geotextile Filter Fabric	88-1.02B	300-8
Perforated Pipe	68-2.02	207-13.4

Note(s):

- Drainpipe should drain to an outlet.
- Filter fabric wraps completely around perforated drainpipe and pervious materials.

Figure 8 - RETAINING WALL DRAINAGE

4.5 SOIL CORROSION POTENTIAL

One soil sample was tested for pH, soluble chloride and soluble sulfate, and soil electrical resistivity for corrosion potential. The test values are summarized in Table 11.

Analytical chemical test results indicated a value of 531.9 parts per million (ppm) soluble sulfate concentration in the near-surface soils. Based on these test results, we recommend that the concrete be designed for exposure class S1 from ACI 318 (ACI, 2011).

The corrosion potential test results are presented in Appendix E. Based on Caltrans Standards (2021) and the chemical test results, the on-site soils are classified as non-corrosive to buried metal pipes. In addition to the soil characteristics, external factors such as nearby active corrosion systems will greatly affect the need for an active corrosion protection system. The test data provided herein can be used by others to develop details of corrosion protection. Borrow soils imported to the Project site should be tested for corrosion potential.

Table 11 - CORROSION POTENTIAL

CONSTITUENT	CRITERIA FOR CORROSIVE MATERIALS	VALUE
pH	<5.5	7.2
Soluble sulfate content (ppm)[1]	>1,500	531.9
Soluble chloride content (ppm)	>500	7.9
Electrical resistivity (ohm-cm)	<1,500	1,541

Note(s):
- Caltrans Corrosion Guidelines (2021)
- ppm = parts per million.
- The lowest values for corrosive materials criteria are presented.

4.6 NOTES FOR CONSTRUCTION

The proposed CIDH piles will extend through gravel/cobble/boulder-rich alluvial dense sands. Additionally, the site is highly variable with layers of boulders, cobbles, and gravel, and those materials can be encountered at any depth. The subsurface cohesionless soils have the potential to slough, cave, and bottom heave during CIDH pile installation when subjected to vibration load from the adjacent traffic or if shallow groundwater is encountered. In addition, loss of drilling fluids was encountered during the subsurface field exploration. Therefore, “wet” construction methods and temporary casings should be considered for ease of construction and to reduce the potential for CIDH pile anomalies. The application of temporary casing may minimize loss of drilling fluid.

When "wet" construction methods are used, the integrity of concrete should be checked using downhole gamma-gamma and/or cross-hole sonic testing; PVC inspection pipes should be installed within the CIDH piles to facilitate the testing. Caltrans Standard Specifications for "Cast-in-Place Concrete Piling" should be followed. Difficult drilling conditions also should be anticipated to penetrate the very dense soils present at the site. In general, a minimum of 24 hours should be allowed between placing concrete in one pile shaft and drilling any nearby shafts or performing any other excavations within four pile diameters. It is the responsibility of the contractor to review all the pertaining boring records and LOTBs to understand the subsurface materials encountered in the borings, to select the appropriate drilling equipment, and to apply their means and methods to drill and install the CIDH piles.

Drilling and casing techniques, such as the oscillator casing method, can also be considered to help reduce construction-induced CIDH structural anomalies. Construction methods will have significant effects on the load-carrying capacity of the installed CIDH piles. Significant quality control and care must be exercised during construction including removal of temporary casing to ensure that the construction methods do not compromise the development of side friction. Selection of the CIDH pile construction contractor should be based on proven performance record on similar projects.