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REVISED FINAL GEOTECHNICAL REPORT

Reds Meadow Road Improvements

MADERA COUNTY, CALIFORNIA

SHANNON & WILSON

January 22, 2021

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Submitted To: Jacobs
2485 Natomas Park Dr, Ste 600
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Attn: Ed Henderson

Subject: REVISED FINAL GEOTECHNICAL REPORT, REDS MEADOW ROAD IMPROVEMENTS, MADERA COUNTY, CALIFORNIA

Shannon & Wilson prepared this report and participated in this project as a subconsultant to Jacobs. Our scope of services was specified in Revision 5 to Agreement Number 101001707 with CH2M Hill, Inc. This report presents our geotechnical design recommendations and was prepared by the undersigned.

We appreciate the opportunity to be of service to you on this project. If you have questions concerning this report, or we may be of further service, please contact us.

Sincerely,

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1 INTRODUCTION

This report provides geotechnical engineering recommendations for the proposed improvements to California Forest Service Road 03S11 (Reds Meadow Road), which is located west of Mammoth Lakes in Madera County, California. The project includes design of the two-lane widening for the upper 2.5 miles of Reds Meadow Road; pavement resurfacing, rehabilitation, and restoration (3R) of the lower 5.8 miles of Reds Meadow Road; and rehabilitation of the Minaret Vista lookout and parking area.

The purpose of this report is to present our geotechnical, pavement, and materials recommendations for the project. The report summarizes our subsurface explorations, laboratory testing, and geotechnical engineering studies, and presents conclusions and recommendations for design and construction of the roadway features based on the 80-percent plan (Jacobs, 2019a), revised 80-percent plan sheets (Jacobs, 2019b, 2020), review comments and resolutions (U.S. Department of Transportation, Federal Highway Administration [FHWA], Central Federal Lands Highway Division [CFL], 2019a,b), and our conversations with Jacobs and CFL.

This report supersedes our previous reports (Shannon & Wilson, 2016, 2018, 2019, 2020).

2 SCOPE OF WORK

We conducted our services in general accordance with our Purchase Order No. 101001707 with Jacobs dated April 26, 2018 and subsequent agreement revision. For the project we:

- managed field preparation, including coordinating with the Forest Service and subcontractors, and completing utility locates
- observed, logged, and collected soil and rock samples from borings and test pits
- mapped surficial geologic materials and rock slopes
- performed a seismic refraction survey
- completed laboratory testing on selected soil and rock samples
- evaluated geotechnical data and completed geotechnical engineering analyses
- completed pavement design
- resolved design team comments
- prepared this report

3 SITE AND PROJECT DESCRIPTION

Reds Meadow Road is in Inyo National Forest, approximately 4 miles west of Mammoth Lakes, California. The project is divided into two parts, the upper 2.5 miles and the lower 5.8 miles. As indicated on Figure 1, the upper 2.5 miles begins on top of the ridge at Minaret Vista Entrance Station and traverses down the mountainside to Agnew Meadow. The lower 5.8 miles winds through the valley floor, beginning at Agnew Meadow and ending at Reds Meadow Resort and Pack Station. We have also included the Minaret Vista lookout and parking lot in the lower 5.8 miles. The existing topography for the upper 2.5 miles is mountainous with steep grade changes on either side of the roadway, while the lower 5.8 miles is generally gentle to hummocky with localized steep grades.

The existing roadway is currently paved with asphalt concrete (AC). The age of the existing AC is unknown. The condition of the pavement is variable, but generally deteriorated with abundant cracking and asphalt patches. Within the upper 2.5 miles, there is typically a longitudinal crack approximately 3 feet from the downslope edge of the pavement. This crack is typically open, vertically offset (outside edge down), and frequently patched (see Figure E-2, Photograph 3). In Appendix E, we provide a summary of our pavement conditions assessment and photographs illustrating the observed conditions.

In the upper 2.5 miles, the project includes realignment and widening of the roadway from the single lane with turnouts that currently exists to two lanes. The widening and realignment are being accomplished using soil nail walls, reinforced soil slopes, and cut and fill slopes. The roadway typical section consists of two 11-foot wide lanes, 1-foot wide shoulders on each side, a 2-foot wide paved gutter and curb on the upslope side, and a 3-foot clear zone on the downslope side (Jacobs, 2019a).

In the lower 5.8 miles, the proposed pavement improvements consist of full-depth reclamation (FDR) and constructing a new hot-asphalt concrete pavement (HACP) on top. Geotechnical mitigation as part of the improvements may also include stabilization of shoulders using reinforced soil slopes (RSS; height ≤ 5 feet) or subexcavation in localized areas to improve poor subgrade conditions.

4 FIELD EXPLORATIONS AND LABORATORY TESTING

We performed our field services in two phases, the first from May 18 through 27, 2018 and the second from October 15 to November 5, 2018. Shannon & Wilson representatives and subcontractors conducted geologic mapping of the surficial units and select rock slopes, completed a pavement condition survey, excavated test pits, completed geotechnical exploration borings, and performed seismic refraction lines. We performed geotechnical laboratory testing on select samples collected during our field services. The following sections describe each of these activities.

4.1 Subsurface Exploration Program

Our subsurface exploration program included excavating 8 test pits, drilling and sampling 51 borings, and performing a seismic refraction survey at 16 locations. We excavated the test pits into the toe of the existing cut or natural slope on the upslope side of the roadway. In the upper 2.5 miles, we drilled the borings approximately 3.5 feet from the outside edge of the existing roadway. In the lower 5.8 miles and at the Minaret Vista lookout and parking area, we drilled the pavement borings in the center of one of the travel lanes.

The subsurface exploration locations are shown on Figure 2. Appendix A discusses the exploration program in further detail, describing the excavation procedure used to complete the test pits, the drilling and sampling procedures used to complete the borings, and presents the seismic refraction survey report. In addition, Appendix A presents the individual test pit and boring logs. Appendix D shows photographs of the test pit and boring locations.

4.2 Geological Mapping

We performed geologic mapping of the upper 2.5 miles of Reds Meadow Road. We observed rock outcrops in both cut slopes and natural slopes along the alignment, typically on the upslope side of the roadway. On the downslope side of the roadway, we primarily observed fill slopes and natural slopes, and occasionally observed a rock outcrop. We mapped the limits of the geologic units and collected descriptions of each unit. We did not perform geologic mapping of the lower 5.8 miles or Minaret Vista lookout and parking area and instead relied on our borings for our interpretation of the subsurface conditions. See Figure 2 for the distribution of the geologic materials along the upper 2.5 miles.

4.3 Rock Slope Geological Mapping

The 30-percent plan field review redline markup included 8 proposed cut slopes, which we designated RW-1 through RW-8 (Jacobs, 2018). Slopes RW-3, RW-5, RW-6, and RW-7 were subsequently removed from the proposed project based on roadway realignments of the design. The 80-percent design submittal plan includes the previously proposed cut slopes RW-1, RW-2 (parts A and B), RW-4, and RW-8 and adds new cut slopes from Stations 34+20 to 34+50 and 95+25 to 98+80 (Jacobs, 2019a). We have designated these new slopes as RW-9 and RW-10.

A Shannon & Wilson representative performed geologic mapping of cells of the existing rock outcrop within or near the limits of the proposed rock cut slopes RW-1, RW-2 (parts A and B), RW-4, and RW-8, which included observation of geologic features and collection of discontinuity data and rock mass information. We did not perform geologic mapping of cells within or near RW-9 and RW-10, because these cut slopes originated after our field work was completed.

We have summarized the rock cut slope designations, mapped cells, and limits of the existing rock outcrops in the following exhibit.

Exhibit 4-1: Rock Slopes

<table>
<tr><th colspan="2">Proposed Rock Cut Slope</th><th></th><th colspan="2">Existing Rock Outcrop</th></tr>
<tr><th>Designation</th><th>Limits (stations)</th><th>Mapped Cells (stations)</th><th>Limits (stations)</th><th>Inclination (degrees</th></tr>
<tr><td rowspan="3">RW-1</td><td rowspan="3">15+15 to 16+50</td><td rowspan="3">14+50 to 15+10</td><td>12+00 to 13+25</td><td rowspan="3">63</td></tr>
<tr><td>14+30 to 16+80</td></tr>
<tr><td>28+20 to 28+90</td></tr>
<tr><td>RW-9</td><td>34+20 to 34+50</td><td>Not Performed</td><td>-</td><td>-</td></tr>
<tr><td rowspan="3">RW-2A and RW-2B</td><td rowspan="3">38+10 to 40+50
41+40 to 47+00</td><td>38+00 to 38+40</td><td>37+80 to 40+80</td><td rowspan="3">70 to 84</td></tr>
<tr><td>41+70 to 42+30</td><td rowspan="2">41+50 to 47+20</td></tr>
<tr><td>43+05 to 43+50</td></tr>
<tr><td rowspan="2">RW-4</td><td>80+20 to 84+45</td><td>81+50 to 83+50
84+00 to 84+50</td><td>79+50 to 84+50</td><td>60 to 78</td></tr>
<tr><td>86+45 to 90+10</td><td>86+50 to 86+80
88+00 to 88+25
88+50 to 89+05</td><td>85+20 to 89+80</td><td>60 to 78</td></tr>
<tr><td>RW-10</td><td>95+25 to 96+80</td><td>Not Performed</td><td>-</td><td>-</td></tr>
<tr><td>RW-8</td><td>132+00 to 135+00</td><td>133+00 to 134+50</td><td>132+00 to 135+50</td><td>70 to 85</td></tr>
</table>

4.4 Seismic Refraction Survey

We performed a seismic refraction survey to assist in determining the rippability of the bedrock and to assist in determining the depth of the bedrock in select locations along the length of the proposed road improvements. The locations of the seismic lines were limited by the steepness of the existing topography, obstructions (e.g. fallen trees), and existing pavement. The seismic line locations are shown on Figure 2 and the seismic refraction survey report is included in Appendix A.

4.5 Laboratory Testing

We completed geotechnical laboratory tests on select samples retrieved from the test pits and borings to estimate soil and rock index properties and engineering properties. Tests included natural water content determination, grain size distribution analysis, Atterberg limits determination, moisture-density relationship (i.e. compaction characteristics), California Bearing Ratio (CBR), and corrosion potential. Appendix B provides discussion of the laboratory test methods performed and provides the laboratory test results. The natural water content, fines content, and Atterberg limits are also shown on the individual boring logs included in Appendix A.

5 REGIONAL GEOLOGY AND SUBSURFACE CONDITIONS

We developed our understanding of the geology and subsurface conditions at the project site based on a review of regional geologic maps and data collected from the surficial mapping and subsurface exploration program.

5.1 Regional Geology

Reds Meadow Road is located on the northwestern side of Mammoth Mountain, a lava-dome complex within California's Sierra Nevada mountain range. Published regional geologic mapping by Huber and Rinehart (1965) indicates the geologic units underlying the Reds Meadow Road consist of surficial deposits, volcanic rocks, granitic rocks, and metamorphic rocks as shown in Figure 3. The hillside terrain in the upper 2.5 miles of roadway is composed of surficial talus and slope wash, andesite volcanic rocks, granitic rocks that include felsic dikes and masses, and metamorphic units that include undifferentiated metavolcanics rocks and calcareous metasedimentary rocks. The lower 5.8 miles of Reds Meadow Road is underlain by surficial deposits that include valley fill and talus and slope wash, andesite volcanic rocks, granitic rocks, and metamorphic rocks that include undifferentiated metavolcanics rocks and undifferentiated metasedimentary rocks.

5.1.1 Surficial Units

The surficial deposits are Quaternary in age and previously mapped as valley fill (Qal) and talus and slope wash (Qts). Valley fill consists of alluvial deposits that vary from fine-grained soil to gravels, including localized zones of appreciable pumice. Talus and slope wash consist of unsorted clay, silt, and gravel, including pumice, with scattered cobbles and boulders. Talus and slope wash appear to originate from volcanic and metamorphic outcrops overlying the roadway.

Fill material is not mapped on published geologic maps, but it is present along the entire alignment in the roadway and is comprised mainly of poorly graded sand with subrounded to subangular gravel.

5.1.2 Bedrock Units

The bedrock geology along the upper 2.5 miles of the Reds Meadow Road is comprised of Triassic-Jurassic age undifferentiated metavolcanics (JTru) and undifferentiated metasedimentary (JTrc) rocks. Undifferentiated metasedimentary rocks consist of shale, marble, siltstone, quartzite, phyllite, and pelite. Metasedimentary fabric consists of all or some of the following: relic bedding to non-bedded and massive, foliated, schistose, crenulated, locally sheared, and glacially polished. Undifferentiated metavolcanics rocks consist of rhyolite-dacite and welded rhyolite tuff. Metavolcanic fabric consists of all or some of the following: phaneritic, porphyritic, foliated, porphyroblastic, and vesicular. Bedrock in the lower 5.8 miles of the Reds Meadow Road are comprised of Triassic-Jurassic undifferentiated metasedimentary rocks (JTrc), Cretaceous age coarse-grained granodiorite (Kcp), and Quaternary age porphyritic andesite (Qad).

5.2 Geographic Setting

The project site is located approximately 4 miles west of the town of Mammoth Lakes, California, in the Inyo National Forest of Madera and Mono Counties. Mammoth Lakes received an average annual precipitation of about 21 inches over the past 20 years (National Oceanic & Atmospheric Administration, 2016). Mammoth Lakes and Reds Meadow Road are considered a snow area with frost depth reported to be approximately 8 inches (U.S. Frost Depth Map, 2018). Mammoth Lakes design guidelines recommends designing for a frost depth of 24 inches (Mammoth Lakes, California 2016).

5.3 Subsurface Conditions

The subsurface conditions at the site can be grouped into three general categories: existing pavement, soil units, and bedrock units. For the soil and bedrock, we correlated the regional geologic units discussed above with the soil and rock samples collected during our exploration program. The following sections provide additional information on the geologic units encountered.

Subsurface explorations indicate that these units are variable in composition and origin, and to some degree, have differing engineering characteristics. We developed 21 cross sections to illustrate the subsurface conditions and to perform engineering analyses for the proposed slopes, RSS, and soil nail walls. These cross sections are presented on Figure 4.

5.3.1 Existing Pavement

We drilled borings within the existing roadway and encountered AC underlain by soil, with no aggregate base course present. The existing AC thickness varied from 1.8 to 4.6 inches. The observed pavement subgrade materials consisted of poorly graded sand with variable amounts of clay, silt, and gravel; silty sand with gravel; and poorly graded gravel with sand, with localized appreciation of pumice. Along the lower 5.8 miles of Reds Meadow Road, our borings occasionally encountered shallow refusal on fresh, high strength granodiorite or andesite beneath the existing pavement.

5.3.2 Surficial Units

The soils we encountered during subsurface exploration included artificial fill (af), valley fill (Qal), and talus and slope wash (Qts). These soils were generally classified as loose to dense, clayey/silty sand with gravel, sand with gravel, clayey gravel with sand, and gravel with sand; and medium stiff to very stiff lean clay with sand and gravel. Along the upper 2.5 miles, the observed thickness of soil extended to 4.5 to 17.5 feet below the existing ground surface (bgs), except for boring SW-B-46 (located at Station 112+50) where soil was encountered to the maximum depth of the boring at 31.5 feet bgs. Along the lower 5.8 miles of road, where the borings were a maximum of 6.5 feet bgs, the observed thickness of soil varied from 2.0 to the bottom of the borings (greater than 6.5 feet). We observed fill thicknesses in the borings that ranged from 0 to 5.0 feet.

The surficial soils we encountered during geologic mapping and subsurface exploration included pumice-rich soil. This soil is lightweight, relative to the non-pumice-rich soil, and has poor strength properties. We have summarized the approximate station ranges of pumice-rich, non-pumice-rich, and zones of interfingering of the two materials (both) in Exhibit 5-1, shown below.

Exhibit 5-1: Pumice-Rich and Non-Pumice-Rich Surficial Soils

Station Range		
Start	End	Soils
10+00	30+00	non-pumice-rich
30+00	50+00	both
50+00	67+00	pumice-rich
67+00	77+00	non-pumice-rich
77+00	87+00	pumice-rich
87+00	95+00	non-pumice-rich
95+00	98+00	pumice-rich
98+00	111+00	both
111+00	129+00	pumice-rich
129+00	136+00	both

5.3.3 Bedrock Units

We observed bedrock in outcrops during our geologic mapping and our subsurface explorations. Rock outcrops were observed in both cut slopes and natural slopes along the alignment. The rock types observed were dark gray non-calcareous relic-bedded shale, gray to white fissile marble, yellow-brown to reddish brown and dark blue relic-bedded non-calcareous quartzite, light gray crenulated and schistose pelite and phyllite, red-brown to light brown porphyritic rhyolite-dacite, and red-brown porphyritic welded rhyolite tuff. The rock ranged from moderately strong to medium high strength, was moderately weathered, and contained very close to wide spaced discontinuities. Observed rock block sizes varied from 1-foot to 5 feet in maximum dimension.

The bedrock we encountered during the subsurface exploration consisted of andesite (Qad), granodiorite (Kcp), undifferentiated metavolcanic (JTru) rocks, and undifferentiated metasedimentary (JTrc) rocks. Along the upper 2.5 miles, we observed bedrock in the borings at depths that ranged from 4.5 feet to greater than the total depth explored of 31.5 feet bgs. Auger refusal was encountered in 23 of the 25 borings along the upper 2.5 miles of road at depths that ranged from 7.7 to 31.5 feet bgs. In most cases, bedrock was identifiable by rock chips recovered during sampling. In borings where we did not obtain a sample after reaching refusal and an outcrop was present at a proximal distance, we assumed bedrock caused the refusal. In borings where we did not obtain a sample and no outcrop was within a proximal distance, refusal may have occurred in these borings on a boulder, cobble, or bedrock. In borings along the lower 5.8 miles of Reds Meadow Road, we observed bedrock at depths that ranged from 2 to 2.5 feet, when encountered. Nearby outcrops were used to quantitatively estimate rock strength which ranged between strong to very strong.

Where we show bedrock in Figure 2, it is overlain by approximately 4 feet or less of surficial soil. This material may consist of weathered bedrock, talus and slopewash, and/or existing artificial fill.

5.4 Groundwater

We observed groundwater within 16 of the 51 borings, which was likely seepage that will fluctuate with snowmelt and rainfall. Within the upper 2.5 miles, we observed groundwater in 4 of the borings, at depths ranging between 4.5 and 17 feet bgs. We observed groundwater along the lower 5.8 miles of the alignment within 12 of the borings, at depths ranging from 1.7 to 5.5 feet bgs. In addition, we observed groundwater at the surficial soil-bedrock contact in 2 test pits along the upper 2.5 miles of the alignment, approximately 2 to 4 feet bgs. We also observed groundwater seeps on the existing surface during geologic mapping of the upper 2.5 miles. The locations of observed seepage on the existing surface are presented on Figure 2.

An affiliate of the Reds Meadow Resort and Pack Station indicated that portions within the lower 5.8 miles of the road alignment overflows with several inches of water each April. This overflow reportedly occurs between stations 613+20 and 615+90, and between stations 757+00 and 760+50.

6 GEOLOGIC HAZARDS

This section identifies potential geologic hazards that exist along Reds Meadow Road, discusses the potential adverse impacts of the geologic hazards, and provides recommendation measures to mitigate these impacts, where required.

Geologic hazards that could potentially impact the project include slope stability (including rockslides, rockfall, debris flows, erosion), corrosion potential, seismic hazards (including fault rupture, seismic ground motion, liquefaction, and seismic compression), and volcanic eruption. We understand CFL's performance expectation for the project is to widen the roadway to address traffic safety concerns while maintaining the existing risk level of geologic hazards. As such, several of the geologic hazards discussed in the following sections do not include mitigation measures. Along the same lines, we did not evaluate geologic hazards for the existing natural, cut, and fill slopes along the road alignment that were not being modified by the proposed design.

6.1 Slope Stability

Potential slope stability hazards include rockslides, rockfall, debris flows, and erosion. These various potential hazards are discussed in the following sections.

It is apparent that rockslides, rockfall, and/or debris flows have occurred above portions of the upper section of the roadway, based on our geologic mapping, borings, oblique view of images in Google Earth, and observations provided by the Forest Service. These events appear to have originated from the volcanic rocks that form the steep topography of the overlying ridge. The mostly treeless, bowl-shaped terrain between the ridge and roadway funnels the debris into a channel that crosses Reds Meadow Road from Stations 104+00 to 106+60 and, to a lesser degree, from Stations 96+60 to 99+40, 114+00 to 116+00, and 127+60 to 128+60. Treeless channels or areas with new growth trees and saplings suggest previous slope failures from above the roadway stripped the trees and transported them downslope.

Mitigation options for slope instability from above the road could include using debris flow and/or rockfall fences, oversizing culverts for debris passage, creating flow deflection or areas for debris deposition, or improved maintenance options for quick recovery following an event. These measures could be implemented later if the performance criteria for the roadway changes.

6.1.1 Rockslides

Rockslides consist of a relatively large mass of rock (or rock and soil) mobilized along a discontinuity in the bedrock. These differ from rockfall events, which consists of individual blocks that originate from outcrops or over-steepened slopes. We are not aware of any recent rockslides that have affected the roadway and, as such, the risk of a large rockslide appears to be low at this time.

6.1.2 Rockfall

Reds Meadow Road is located beneath natural cliffs and steep slopes with a rockfall hazard. There are indications of rockfall events as evidenced by the topographic expression of the source areas and boulders resting on the slope above and below the roadway in the upper 2.5 miles. However, we did not observe scars from rockfall within the roadway. In addition, based on conversations with Forest Service maintenance personnel, rockfall has not impacted the road. While we cannot dismiss the potential for rockfall hazard to impact the roadway, it does not appear to be a frequent occurrence.

In addition to naturally occurring rockfall hazards, rockfall may also originate from new rock cut slopes. Rockfall hazards are increased where cuts expose weak or unfavorably jointed rock or undercut surficial soils such as slope wash and talus. While the slope excavation angles can be designed to reduce rockfall hazard, weathering action on slopes may increase the rockfall hazards with time. Appropriate maintenance operations may be required to manage these hazards, as discussed in the Rock Cut Slopes section below.

6.1.3 Debris Flows

A debris flow is a viscous, flowing mixture of water, mud, rock fragments, and organic debris transported down slope during heavy rainfall or snowmelt runoff events. Forest fires can be a contributing factor to an increased frequency of debris flows. Debris-flow deposition areas, or debris fans, form through repeated debris flows primarily at the base of drainages or debris flow channels. Large flow volumes and boulders incorporated into a flow can bury a roadway and cause damage to structures.

Evidence of previous debris flows crossing the roadway exist between Stations 104+00 and 106+60 and extending to the lower section of the roadway between Stations 548+25 and 552+00. Also, two debris flows occurred on October 16, 2016 during heavy rains near Stations 105+00 and 503+00. The events consisted of sediment and storm water runoff concentrating in natural drainage channels, overwhelming the existing drainage system, and covering the roadway. Similar debris flow hazard areas are present through the Reds Meadow Road alignment where it crosses natural drainages. Potential mitigation measures for debris flows are discussed above.

6.1.4 Erosion

Typically, sandy soil on steep slopes subject to high velocity water flow or non-vegetated areas are susceptible to erosion. Existing cut slopes (except for pumice) appear to be performing well, although raveling is present in some locations where surficial units appear to be eroding during periods of inclement weather. Slopes that expose surficial units are susceptible to shallow slumps where over-steepened, which leads to sedimentation within the drainage ditches and clogging or blockage of culverts. Forest Service maintenance personnel indicated several culverts need to be cleaned and drainage ditches re-established along the upslope side of the roadway at the beginning of each season.

6.2 Corrosion Potential

Soil can be corrosive when in contact with concrete or steel pipes and other buried metal elements. To assist in estimating the corrosion potential across the project, we tested 11 boring samples and 4 test pit samples for chloride concentration, water soluble sulfate concentration, resistivity, and pH. Discussion of these parameters, the testing procedures performed, and results are provided in Appendix B.

The concentration of water-soluble sulfates and chlorides measured in the samples was 31 to 58 parts per million. Based on classifications as defined by ACI-318-19 (ACI, 2019), these test results suggest an exposure class of S0 for concrete exposed to site soils.

The resistivity measured in the samples ranged from 4,988 to 46,994 ohm-centimeters. Based on correlations developed by Roberge (2012), these values suggest essentially noncorrosive to corrosive subsurface conditions for metal in contact with subsurface materials across the site.

The pH test results indicated a normal range for soils in terms of corrosion potential.

The test results and the above discussion are provided to assist the designer in the selection of project materials, concrete type, or other features with respect to corrosion. As appropriate, the designer should consider protective measures, such as coatings, upsizing for section loss, or using alternative materials to reduce the corrosion potential.

6.3 Seismic Hazards

The project is located in a seismically active region of California and is expected to experience the effects of future earthquakes on active faults located within the region. Seismic hazards that may potentially impact the proposed improvements include fault rupture, seismic ground motion, seismic settlement (which includes liquefaction and seismic compression), and seismically induced slope stability. We provide a discussion of each of these hazards in the following sections. However, consistent with CFL's decision to maintain the current level of risk, we understand these geologic hazards will not be mitigated as a part of this project.

6.3.1 Fault Rupture

Surface fault rupture hazard evaluation is based upon criteria developed by California Division of Mines and Geology (CDMG), now known as the California Geologic Survey (CGS), for the Alquist-Priolo Zone Act program (CGS, 2018). Based on the CGS criteria, an active fault is a fault that has ruptured within the Holocene geologic time period (about the last 11,700 years). According to the United States Geological Survey (USGS) online mapping tool, active faults do not cross the project alignment. However, there are mapped faults near the project that include the Hartley Springs Fault Zone and the Hilton Creek Fault Zone (USGS, 2006). According to USGS reports, these fault zones have experienced Quaternary movement.

The Hartley Springs Fault Zone is mapped approximately 1 to 3 miles east of the Minaret Vista entrance station (Sawyer and Bryant, 1995). It is approximately 14 miles long with escarpment heights of 2,000 feet and is comprised of mostly northwest trending faults (Bailey, 1989). The Hilton Creek fault zone is mapped at least 7½ miles to the east and was responsible for several events in 1980 with moment-magnitudes greater than 6.0 (Sawyer and Bryant, 1995; Taylor and Bryant 1980). It is approximately 18.5 miles long with escarpment heights of nearly 5,000 feet and is comprised of mostly northwest trending faults (USGS, 2016). In the past 80 years the region has experienced 33 earthquakes of moment-magnitude 5.0 or greater (USGS, 2016).

Considering there are no documented faults with Quaternary movement that cross Reds Meadow Road, it is our opinion the potential for surface rupture is low.

6.3.2 Seismic Site Classification

Site class depends on the type of rock and the thickness of overburden deposits. Based on the results of our subsurface exploration, we recommend using Site Class B for the earth retaining structures located in the upper 2.5 miles.

6.3.3 Seismic Ground Motion

The road alignment will likely experience seismic ground shaking during an earthquake along faults in the region. Earthquakes along several active faults in the region could cause moderate to strong ground shaking. The intensity of earthquake motion will depend on the characteristics of the generating fault, distance to the earthquake fault, earthquake magnitude, earthquake duration, and site-specific geologic conditions. Ground motions may be amplified or attenuated by soil deposits at the site depending on the level of ground shaking in the underlying bedrock, soil type/density, depth to bedrock, and other factors.

According to the Federal Lands Highway Project Development and Design Manual (PDDM) (U.S. Department of Transportation [USDOT] and FHWA, 2018) and supplemental Technical Guidance Manual (TGM; USDOT and FHWA, 2007), geotechnical seismic design should be consistent with the philosophy for structure design, which state that loss of life and serious injury due to structure collapse are minimized to the extent possible and economically feasible.

Seismic hazards are evaluated considering two different design events:

- 10 percent probability of exceedance in 50 years, or 475-year event (USDOT and FHWA, 2018)
- 7 percent probability of exceedance in 75 years, or 1,000-year event (AASHTO, 2017)

We determined ground motion parameters for the project site using the USGS Unified Hazard Tool (USGS, 2017). The following exhibit presents recommended seismic design ground motion parameters for the upper 2.5 miles of Reds Meadow Road.

Exhibit 6-1: Seismic Design Ground Motion Parameter

Ground Motion Parameters / Coefficients	Return Period 475-Year	Return Period 1,000-Year
Site Class	B	B
Peak Ground Acceleration (PGA) (g)	0.26	0.34
Short-period Spectral Acceleration, S_S (g)	0.55	0.74
Long-period Spectral Acceleration, S_1 (g)	0.12	0.17
Site Factor, F_{pga}	1.00	1.00
Site Factor, F_a	1.00	1.00
Site Factor, F_v	1.00	1.00
Peak Design Spectral Acceleration, A_S (g)	0.26	0.34
Short-period Spectral Acceleration Adjusted for Site Class Effects, S_{MS} (g)	0.55	0.74
Long-period Spectral Acceleration Adjusted for Site Class Effects, S_{M1} (g)	0.12	0.17
Short-period Design Spectral Acceleration, S_{DS} (g)	0.37	0.49
Long-period Design Spectral Acceleration, S_{D1} (g)	0.08	0.11

Notes:
g = gravity

6.3.4 Seismic Settlement

Seismically induced settlement is divided into two parts, soil above and below the groundwater table. Settlement below the water table is associated with liquefaction of loose, saturated, granular soil due to the seismic loading. Settlement above the water table is associated with seismic compression of loose, granular soil during seismic ground shaking. Based on the site geologic conditions and the design Peak Ground Acceleration (PGA), it is our opinion there is a negligible potential for liquefaction and seismic compression to impact the upper 2.5 miles of the roadway and a low to moderate potential for the lower 5.8 miles.

6.3.5 Seismically Induced Slope Stability

The risk for slope instability increases during earthquakes, which may result in seismically induced landsliding, surficial failures, rockfalls, and rockslides. Considering the estimated design PGA for the site, steep nature of the overlying slopes and numerous rock outcrops, it is our opinion there is the potential for seismically induced slope instability to impact the proposed project. Portions of the existing slopes may have a factor of safety (FS) near 1.0 in the present condition. As a result, minor earthquake induced shaking is assumed to produce ground accelerations that may mobilize some material. See Section 7.0 for additional discussion.

6.4 Volcanic Eruption

According to the USGS (2016), Reds Meadow Road is in an area of past volcanic activity. It is at least ¾ of a mile west of the Long Valley Caldera, a basin-shaped volcanic depression that was formed when it erupted approximately 760,000 years ago. Smaller eruptions and uplift of the magma chamber occurred periodically over the next several hundred thousand years. The Devil's Postpile eruptions and flows occurred approximately 100,000 years ago and repeated eruptions between 110,000 and 56,000 years ago formed Mammoth Mountain. The area has experienced increased volcanic related activities in the past few decades including earthquakes, ground uplift, and gas emissions.

Volcanic eruption and the consequences are beyond the standard of practice for civil engineering projects at this time and location. As such, and in line with CFL's desire to maintain the existing hazard level, we have not provided potential mitigation measures.

7 GEOTECHNICAL RECOMMENDATIONS

Engineered slopes, RSS, and soil nail walls are planned to accommodate the proposed roadway widening and realignment of the upper 2.5 miles. The appropriate type of system at any location depends on the existing topography, geology, and project constraints. We understand that a 13-foot wide travel lane must be maintained during construction, with limited periods of full roadway closures.

7.1 Pumice

We observed areas that were dominated by gravel-sized particles of pumice (i.e., pumice-rich soil) while performing geologic mapping and subsurface exploration. These locations are summarized in Exhibit 5-1 presented in Section 5.3.2.

Pumice is a cellular volcanic rock with a relatively low unit weight and poor strength characteristics. We observed severe levels of pavement distress, including subgrade failures and fatigue cracking and rutting, in areas where pumice was present. SPT blow counts in pumice-rich materials ranged from 2 to 4 blows per foot.

We recommend pumice-rich soil not be used for direct support of fill slopes, RSS, or improvements where dense and unyielding subgrade cannot be achieved. In these areas, we recommend a subexcavation.

We understand CFL has had success breaking down the gravel-sized particles of pumice into a well graded soil, by crushing it with a bulldozer, that can be used for fill material. If this method is used, we recommend performing field and laboratory testing of the pumice-derived fill soil to evaluate its engineering properties.

7.2 Engineered Slopes

Fill and soil cut slopes are proposed at gradients of 1V:2H to a maximum height of approximately 30 feet and 1.5 feet, respectively (Jacobs, 2019a,b, 2020). Rock cut slopes are proposed at a gradient of 1V:0.5H to a maximum height of approximately 22 feet.

We evaluated the proposed fill slopes to determine if they exceed the CFL-required static FS of 1.3 (USDOT and FHWA, 2018). The PDDM indicates standard practice is that seismic analysis is not performed on slopes, except where ground anchors or stabilization structures are used. Considering this, we did not perform seismic slope stability analyses of the proposed cut or fill slopes.

The proposed cut slopes in soil are located on the upslope side of the roadway, behind the proposed curb. Considering their exposed height is 1.5 feet or less, we did not evaluate the global stability of these soil slopes.

7.2.1 Fill Slopes

Fill slopes are planned at a gradient of 1V:2H to heights of 30 feet of less. For our analyses, we assumed a fill material with an angle of internal friction of 34 degrees and placement is in accordance with FP-14 U.S. Customary Units (USDOT and FHWA, 2014) standards (FP-14). Based on our analyses, the proposed fill slopes meet the CFL-required static FS with a minimum key width of 15 feet. For fill slope heights of 20 feet, or less, the key width may be reduced to 10 feet.

The base of the fill slope should be keyed into the existing slope surface and the fill benched into existing native soils during filling, in accordance with the requirements provided in FP-14 (USDOT and FHWA, 2014). We recommend the key be tilted into the slope at 2 percent and placing a subsurface drain in the heel of the key to avoid building up hydrostatic pressure. Details regarding subsurface drain are provided below in the Drainage section. We anticipate fill slopes will be difficult to vegetate and likely be subject to surficial deterioration and ongoing maintenance. We recommend considering temporary erosion control blankets or permanent turf reinforcement mats and hydroseed to vegetate the fill slopes.

7.2.2 Rock Cut Slopes

We evaluated rock cut slopes for kinematic stability at the five proposed rock cut locations (RW-1, RW-2A, RW-2B, RW-4, and RW-8) shown in the 30-percent design plans. We evaluated rockfall catchment for five cut slope inclinations provided in the Rockfall Catchment Area Design Guide published by FHWA (Pierson and others, 2001) and three catchment ditch configurations.

The 80-percent design plans have changed the extents of rock cut slopes RW-1, RW-2A, RW-2B, RW-4, and RW-8, and added two new slopes, RW-9 and RW-10. The extents of slopes RW-1, RW-2A, RW-2B, and RW-4 have been reduced, while the extents of RW-8 been expanded on the southeastern end by 23 feet. The extents of the rock cut slopes are summarized in Exhibit 4-1.

The rock slopes that we did not previously evaluate, which include the expanded southeastern end of RW-8, RW-9, and RW-10, may expose surficial soil or bedrock, depending on the thickness of the surficial soil and depth of cut. We should observe these locations during construction to confirm bedrock is exposed at the finished slope face and perform the cell mapping. We will use the data collected to perform the kinematic analyses described below and provide mitigation measures, if needed.

Where temporary cuts for soil nail walls expose bedrock, these walls may potentially be changed to rock cut slopes. As above, we should observe these locations to perform cell mapping and kinematic analyses so that we could recommend the appropriate slope inclination and ditch width.

We did not evaluate the existing cut or natural rock slopes that are not being modified by the proposed construction. This is consistent with CFL's performance expectation for the project which is to maintain the existing risk level.

7.2.2.1 Geomechanical Rock Mass Classification

We completed geomechanical rock mass classifications using the field data collected following the Rock Mass Rating System (RMR) by Bieniawski (1989) and the Geological Strength Index from Hoek and Bray (1997).

The RMR, also referred to as the geomechanics classification system, is based on the algebraic sum of six rock mass property ratings: strength of intact rock material, rock quality designation, spacing of discontinuities, condition of discontinuities, groundwater conditions, and orientation of discontinuities relative to the excavation or rock slope. To estimate the RMR, we compared field data to published tables by Bieniawski (1989). Values for RMR can range from 0 to 100.

In Appendix C, we provide summaries of the geomechanical rock mass information collected during our field activities as part of the Rock Slopes calculation package summaries and results.

7.2.2.2 Rockfall Catchment

We used the rockfall slope charts presented in the Rockfall Catchment Area Design Guide published by FHWA (Pierson and others, 2001) to evaluate ditch widths and configurations assuming 80-percent of the rocks will be retained (pers. comm., CFL, 2017). We considered a 40-foot high slope with cut slope inclinations of near vertical (80 degrees), 1V:0.25H, 1V:0.5H, 1V:0.75H, and 1V:1H. The slope charts provide ditch configurations at 1V:4H, 1V:6H, and flat. The shortest slope chart slope height is 40 feet, while most of the cut slopes along the alignment are less than 10 feet high. To relate the 40-foot high slope to the 10-foot high slope, we reduced the slope chart recommended ditch width by 75-percent (to 25-percent of the chart width). For a 20-foot high slope, we reduced the slope chart recommended ditch width by 50 percent to obtain 80-percent retention of rockfall. We provide the rock slope charts with 80-percent catchment, varying ditch configurations, and slope inclinations as part of the Rock Slopes calculation package summary and results in Appendix C. We provide ditch width alternatives for 1V:0.5H rock cut slopes in Exhibit 7-1.

Exhibit 7-1: Summary of Ditch Width Values

Slope Inclination	Ditch Configuration Width for 80% Catchment (feet)		
	1V:4H	1V:6H	Flat
1V:0.5H	2.3	3.8	7.5

Maintenance operations should routinely remove rocks deposited in the ditches to sustain the ditch functionality.

7.2.2.3 Kinematic Stability

To evaluate potential rock cut slope inclinations, we used the discontinuity data we collected to construct pole plots on equal area stereonets using the program Dips (Rocscience, 2018a). Where the stability of a rock cut is controlled by the structure of the rock mass, a kinematic analysis is used to estimate the kinematic potential for rock blocks to move out of the slopes. We analyzed three types of failure modes: planar, wedge, and toppling. The information required to perform an analysis are the design slope dip and dip direction, the orientation of the discontinuities within the rock mass, and the friction angle of the discontinuities.

A kinematically admissible planar failure occurs when the dip of the slope exceeds the dip of the potential slip plane, the potential slip plane daylights the slope, the dip of the plane exceeds the friction angle of the rock, and the dip direction of the sliding plane falls within ± 20° of the dip direction of the slope face.

A wedge failure is similar to a planar failure except the sliding occurs on two planes. A wedge failure is identified when the criteria for a planar failure is met; however, wedge failures may fall outside of the ± 20° of the variation defined within a planar failure since the sliding direction is defined by the line of intersection.

The third type of failure is toppling, which are generally associated with blocks of rock formed by discontinuities with a high angle joint at the slope face. For the purposes of this analysis, we evaluated toppling. Toppling is considered feasible when planes can slide with respect to one another and the direction of the slip planes lies within ± 30° of the dip direction of the slope (Goodman, 1980).

A kinematic analysis does not consider a cohesion intercept when modeling the strength of discontinuities. This method also assumes that the discontinuities are continuous and through going with no “bridging” within the discontinuity. The effect of “bridging” would allow a tensional component (or cohesion intercept) of discontinuity strength. We assumed the rock discontinuity friction angle based on the geomechanical information that we collected in the field, experience with similar rock types, and guidance from the Rock Slopes Reference Manual (Munfakh and others, 1998). We assumed a friction angle on discontinuities of 34 degrees for slopes RW-1, RW-2A, RW-4 and RW-8. For slope RW-2B, we assumed a friction angle on discontinuities of 27 degrees to account for the different rock type.

Our recommended cut slope angles are summarized in Exhibit 7-2. Appendix C provides the calculation for Rock Slopes and presentation of the kinematic analysis.

Exhibit 7-2: Summary of Cut Slope Angles Evaluated

Slope Number	Proposed Rock Cut Slope Station Range	Proposed Rock Cut Slope Maximum Height (feet)	Dip Direction (degrees)	Recommended Slope Inclination
RW-1	15+15 to 16+50	6	157	1V:0.5H
RW-2A	38+10 to 40+50	7	207	1V:0.5H
RW-2B	41+40 to 47+00	9	Varies over length	1V:0.5H
RW-4	80+20 to 84+45 86+45 to 90+10	22	Varies over length	1V:0.5H
RW-8	132+00 to 135+00	18	Varies over length	1V:0.5H

The rock cut slope inclinations presented in Exhibit 7-2 do not remove all kinematically admissible failure types. The inclinations were selected if a failure mode was observed as having the same percentage of dip vectors fall within a critical zone at a lower inclination (i.e., 53 degrees) value as the steeper inclination value (i.e., 63 degrees). The cut slope inclination of 1V:0.5H is generally flatter than the existing rock slope inclinations observed in the field. The existing rock slope inclinations vary from about 60 degrees to near vertical and the reduction in cut slope inclination reduces the kinematically admissible failures observed during the stereonet analysis.

For all slopes there is potential for planar failures to occur. Slope RW-4 was modeled using the program Slide (Rocscience, 2018b) for a 17-foot-tall rock cut at a 1V:0.5H inclination. Within the program, we used anisotropic strengths to model a planar failure that may occur out of the slope. The program indicated that the FS was less than 1.0 for a large, planar failure through the rock slope. Given the moderately to highly fractured nature of the slopes however, a planar failure still has a high likelihood of occurring, but the scale of a planar feature would be limited to the fracture spacing and likely produce rocks about 1 to 5 feet in dimension, as was observed during our geological mapping. To reduce the potential for kinematically admissible planar features to form, the rock cut slopes would need to be cut at an angle equal to or flatter than the exposed planar feature. The angle of the planar features observed during mapping varies from 27 to 49 degrees. The likelihood of planar features and dip vectors observed within a critical zone on the stereonet varies depending on the slope dip direction. During construction, the excavation of cut slopes should be observed closely and modified for slope inclination or consider additional support (i.e., rock bolts or mesh) if planar features appear unstable and unsatisfactory to CFL.

Wedge type kinematically admissible failures identified on the stereonets were further evaluated in the program SWedge (Rocscience, 2017). SWedge allows the user to evaluate two specific joints in relation to the rock cut slope dip and dip direction. Evaluation of the two joints with the slope orientation provides an estimate if the wedge observed on the stereonet forms in real space. The program also allows for scaling of the joints. The scaling feature is useful for modeling fractured rock slopes that have other discontinuities truncating the length of an observed feature. The SWedge modeling indicated that wedges are likely to form at slopes RW-1, RW-2A, RW-2B, and within the first portion of slope RW-4. The formation of wedge failures occurring at the rock slopes has a moderate to high likelihood. However, the size of the features may appear more like rockfall given the fracture spacing observed on the slopes and will likely be about 1 to 5 feet in dimension. Rockfall or wedge failures smaller than 2 feet are anticipated to be contained within the ditch. Rocks larger than 2 feet will spill onto the traveled roadway as they would be larger than the anticipated ditch width.

Toppling failures are not likely to occur at rock slopes RW-1 and RW-8, where toppling was not observed as kinematically admissible and is therefore to have a low likelihood for toppling to occur. Toppling failures at slopes RW-2A, RW-2B, and RW-4 has a moderate likelihood to occur and will likely be limited to the rock size of about 1 to 5 feet as observed during our geological mapping.

7.3 Reinforced Soil Slopes

We performed engineering analyses in accordance with the PDDM. The purpose of our analyses was to determine if the proposed design satisfies the required FS values for global and internal stability.

For our analyses, we assumed the following:

- Finished slope gradient = 1V:1H or flatter, as illustrated on the roadway cross sections provided
- Temporary backcut gradient = 1V:1H or flatter and parallel to the finished slope gradient as illustrated on the roadway cross sections provided
- Reinforcement:
  - Type II grid:
    - Nominal long-term strength = 1,500 lb/ft

- Primary Layers:
  - Vertical Spacing = 2 feet
  - Length = provided in Exhibit 7-3 below
- Secondary Layers:
  - Spacing = mid-height between primary reinforcement
  - Length = 5 feet (does not include wrapping the slope face)
- Wrap the finished slope face by at least 3 feet for both primary and secondary layers

- 250 psf traffic surcharge along the roadway width for static conditions only
- Drained conditions
- RSS excavation is prepared in accordance with the Subgrade Preparation and Proof Rolling section below

We anticipate the RSS finished slope faces will be difficult to vegetate and likely be subject to surficial deterioration and ongoing maintenance. We recommend considering temporary erosion control blankets or permanent turf reinforcement mats and hydroseed to vegetate the RSS.

Our stability evaluation process and results are discussed in the section below.

7.3.1 Stability Evaluation

Our stability evaluation consisted of a two-dimensional, limit equilibrium slope stability analyses using the GeoStudio computer program SLOPE/W (Geo-Slope, 2019), considering both static and seismic conditions. We analyzed potential failure surfaces using the Morgenstern-Price method. The analyses were performed considering circular failure surfaces to determine the critical failure surface (i.e. the failure surface which resulted in the minimum computed FS).

We evaluated both global (or external) stability (which assumes the RSS fill acts like a rigid block and only considers failure surfaces which extend behind the reinforced zone) and internal stability (which considers failure surfaces passing through the reinforced zone). The internal stability evaluates the strength capacity of the reinforcement grid by considering both the tensile capacity and pullout resistance.

For our stability evaluation, we considered a slip surface beginning at the proposed road elevation and ending downslope of the wall. To prevent "infinite slope" condition (i.e. a failure plane which runs parallel to the surface of the slope and extends infinitely down), the downslope search limit was constrained below the base of the RSS fill to a vertical distance equal to the height of the reinforced zone.

The PDDM refers to FHWA's Geotechnical Engineering Circular (GEC) No. 11 (Berg and others, 2009) for guidance for RSS design. GEC No. 11 recommends a minimum FS of 1.3 for static conditions and 1.1 for seismic conditions.

We evaluated seismic slope stability using pseudo-static analysis with an applied horizontal seismic coefficient (k_h). GEC No. 11 recommends using k_h equal to one-half of the design PGA, considering a 7 percent probability of exceedance in 75 years (i.e. a 1,000-year event). The California Department of Transportation (Caltrans) recommends using k_h equal to one-third the design PGA, but not greater than 0.2 g (Caltrans, 2014). Based on discussions with CFL, we understand using the seismic coefficient value in accordance with Caltrans guidance meets CFL's performance expectation for the project. As such, our pseudo-static analysis uses k_h equal to one-third of the PGA, assuming a 1,000-year design event.

7.3.2 Stability Evaluation Results

Based on the results of our analyses, Exhibit 7-3 summarizes reinforcement lengths and reinforced foundation locations. Graphical results of our stability analyses are provided in Appendix C.

Exhibit 7-3: Reinforced Soil Slope Recommendations

Stations		Maximum Height	Reinforcement Length	Reinforced
Begin	End	(ft)	(ft)[1]	Foundation[2]
15+20	134+70	5	10	No
15+20	21+30	11	16	No
22+45	50+30	10	16	No
		11	16	Yes
54+80	68+70	6	16	No
		16	24	Yes
80+80	96+95	8	14	No
96+95	97+25	12	22	Yes
97+25	101+50	8	14	No
		14	18	Yes
		17	22	Yes
101+50	121+30	6	12	No
		10	14	No
129+10	134+70	10	16	No
		12	20	Yes

NOTES:

1 Reinforcement length does not include wrapping the slope face.

2 See the Reinforced Foundation – Unsuitable Fill Slope and RSS Subgrade section for recommendations

If bedrock is present within the temporary backcut, the excavation can follow the rock surface and the primary reinforcement length can be reduced. In this case, the base of the reinforcement should overlap the soil-bedrock contact by at least 1 foot. However, in no case should the primary reinforcement be less than 0.6 times the RSS height.

7.4 Soil Nail Walls

There are 14 soil nail walls proposed in the upper 2.5 miles (Jacobs, 2019a,b; 2020). The walls vary in length, height, backslope angle, and retained soil properties. We developed project-wide design recommendations and performed wall specific global slope stability calculations to determine the soil nail lengths and spacing. We understand the project structural engineer will design the soil nail wall facing.

7.4.1 Design Recommendations

We developed our soil nail wall design recommendations for soil nail pullout resistance, tensile forces, and soil nail bar properties in accordance with GEC No. 7 (Lazarte and others, 2015). We estimated the soil nail pullout resistance, assuming a 6-inch diameter hole drilled using rotary drilling methods. In addition to calculating the tensile force at the nail head (T_0) and the maximum nail tensile force (T_{max}) in accordance with GEC No. 7 section 5.2.1, we performed slope stability analyses to back calculate the forces. We used T_{max} to determine the appropriate bar size and grade. We have summarized our recommendations in Exhibit 7-4.

Exhibit 7-4: Soil Nail Wall Design Parameter

Design	Parameter		Value
Soil parameters	Angle of internal friction (degrees)		30
	Cohesion (psf)		0
	Unit weight (pcf)		90
	Active earth pressure coefficient		0.8
Soil nails	Nominal pullout resistance (kips/ft)	Soil	4.5
		Bedrock	36.0
	Bar designation		#8
	Bar yield stress (ksi)		75
	Bar dip below horizontal (degrees)		15
Tensile forces	T_0 (kips)	Static	12.1
		Seismic	17.6

NOTES:
psf = pounds per square foot
pcf = pounds per cubic foot

pcf = pounds per cubic foot
kips/ft = kips per linear foot
ksi = kips per square inch

The actual grout-to-ground bond strength developed in the field will depend on the contractor's installation methods, particularly the drilling and grouting procedures. The soil nail contractor will ultimately be responsible to achieve the design bond strength value in the field by load testing, as described in the Soil Nail Walls Testing section below.

Hollow-bar soil nails may be appropriate for use in lieu of the solid bar soil nails specified. If hollow-bar soil nails are considered, the contractor should provide a submittal that specifies the materials and installation process that will be used to obtain the nominal pullout resistance, demonstrate the corrosion protection measures are appropriate for the site conditions, and describe the verification and proof testing procedures that will be used.

7.4.2 Stability Evaluation

Our global stability evaluation consisted of a two-dimensional, limit equilibrium slope stability analyses using the GeoStudio computer program SLOPE/W (Geo-Slope, 2019), considering both static and seismic conditions. We analyzed potential failure surfaces using the Morgenstern-Price method. The analyses were performed considering circular failure surfaces to determine the critical failure surface (i.e. the failure surface which resulted with the minimum computed FS).

Considering the variability of subsurface conditions, we analyzed the soil nail walls assuming bedrock is not present. This condition is conservative where bedrock is encountered in the drilled holes. Our estimation of the location of bedrock at select locations is illustrated in the cross sections provided in Figure 4.

We evaluated global stability where the potential slip surfaces extend beyond the end of the soil nails and internal stability where the potential surfaces intersect the soil nails. For our stability evaluation, we considered a slip surface beginning above the proposed wall and ending downslope of the wall. To prevent "infinite slope" condition (i.e. a failure plane which runs parallel to the surface of the slope and extends infinitely up and/or down), the search limits were constrained to a horizontal distance of 3 times the maximum height of the wall both above the top of the wall and also below the roadway. We did not analyze construction stages, which should be evaluated by the contractor.

The stability analyses required a minimum FS of 1.35 for static conditions and 1.1 for seismic conditions. We performed our analyses considering a horizontal spacing of 5 or 10 feet (depending on the maximum wall height) and adjusted the number of rows, vertical spacing, and soil nail lengths to achieve the required minimum static and seismic FS.

We evaluated seismic slope stability using pseudo-static analysis with an applied horizontal seismic coefficient (k_h). We developed k_h following design criteria specified by GEC No. 7 (Lazarte and others, 2015) for design guidance for proposed soil nail walls. GEC No. 7 recommends using k_h equal to one-half of the design PGA, considering a 7 percent probability of exceedance in 75 years (i.e. a 1,000-year event).

7.4.3 Stability Evaluation Results

We have summarized the number of rows, nail length, and horizontal spacing in Exhibit 7-5. Graphical results of our stability analyses are provided in Appendix C.

Exhibit 7-5: Soil Nail Wall Recommendations

Wall	Maximum Height (feet)	Stations		Number of Rows	Soil Nail Length (feet)
		Begin	End		
27	9	24+10	29+45	2	15
34	10	31+15	37+25	2	15
48	8	47+00	48+84	2	15
50	7	49+56	50+34	2	15
62	11	55+05	68+54	2	17
85	10	84+45	85+50	2	15
92	9	91+20	92+14	2	15
93	7	92+35	92+88	2	15
95	12	94+70	95+26	2	20
99	6	98+56	99+02	1	25
100	10	99+26	100+00	2	20
	7	100+00	101+52	1	25
111	9	110+10	111+86	1	15
119	10	116+30	121+33	2	15
131	8	130+25	132+16	1	15

The soil nail lengths in Exhibit 7-5 were determined assuming bedrock is not encountered in the drilled hole. Where bedrock is encountered, the soil nails can be shortened to provide a minimum of 3 feet into bedrock and a minimum total length of 10 feet.

Where one row of nails is recommended, the nails should be located at approximately mid-height of the back of the wall. The nails should be staggered approximately 6 inches vertically. Where two rows of nails are recommended, the rows should be spaced approximately 4 feet vertically and staggered. The top rows of nails should be located 2 feet below the top of the wall, measured where the back of the wall intersects the existing grade.

The bottom row of nails should be located 2 feet above the bottom of the wall to allow for nail installation.

7.4.4 Drainage Considerations

Our analyses assumed drained conditions. To avoid the buildup of hydrostatic pressure, we recommend using drainage composite strips behind the wall. Strips should be located between each nail and connected to an outlet at the base of the wall.

Surface drainage behind the soil nail wall is important to avoid the intrusion of water behind the wall. We recommend providing a non-erosive drainage path behind the wall to transport snowmelt and storm water away from the wall. We anticipate this device will fill with sediment and require maintenance to maintain its function.

7.4.5 Corrosion Protection

Based on the chemical test results for pH, resistivity, and concentrations of sulfates and chloride, the site soils have a non-aggressive corrosion potential (Elias and others, 2009). Corrosion protection should be selected based on the corrosion potential and risk tolerance of the owner.

7.5 Subexcavation – Unsuitable Pavement Subgrade

In areas of unsuitable pavement subgrade, we recommend performing subexcavation to repair the subgrade. In these areas, the existing roadway embankment will be subexcavated to a depth of 2 feet below the bottom of the pavement section and a stabilization geogrid meeting the requirements of FP-14 Section 714.03, placed at the bottom of the subexcavation. The subexcavation should be backfilled using Unclassified Borrow material meeting FP-14 specification 704.06. However, we recommend the Unclassified Borrow material have a maximum particle size of 3 inches.

For planning purposes, we identified areas of existing pavement during the 70-percent field review that have severe rutting. We recommend a subexcavation at the locations tabulated in Exhibit 7-6.

Exhibit 7-6: Subexcavation Locations

Station			
Begin	End	Lane	Underdrain
8+00	10+00	Both	No
55+00	61+00	North bound[1]	No
80+00	89+00	North bound[1]	No
512+20	514+00	North bound	No
577+50	579+25	Both	No
588+00	589+10	Both	No
614+00	615+80	Both	Yes
639+00	641+00	Both	No
641+00	641+60	North bound	No
643+60	645+50	Both	No
647+00	647+60	South bound	No
647+60	649+50	Both	No
736+50	737+20	North bound	No
756+25	759+00	North bound	Yes
768+00	770+90	North bound	No

NOTES:

1 Extend subexcavation to RSS.

In two locations, we observed evidence of groundwater piping and subgrade failures below the existing pavement during our field services in May 2018. These locations were from approximately Stations 614+00 to 615+80 and Stations 756+25 to 759+00. We anticipate these locations will require subexcavation and an underdrain. We recommend installing an underdrain on the upslope side of the roadway, placing a woven, Class 1, Type B geotextile on the bottom of the excavation, and backfilling the excavation using Unclassified Borrow material meeting FP-14 specification 704.06. However, we recommend the Unclassified Borrow material have a maximum particle size of 3 inches.

7.6 Reinforced Foundation - Unsuitable Fill Slope and RSS Subgrade

In all locations to receive fill or structural improvements, the subgrade should be prepared in accordance with the Subgrade Preparation and Proof Rolling section below. If the contractor is unable to achieve a dense and unyielding condition, we recommend constructing a 2-foot deep reinforced foundation zone.

The location of reinforced foundations needed for global stability of the RSS is summarized in Exhibit 7-3. All other RSS subgrades and all fill slope subgrades should be evaluated during construction, in accordance with the Subgrade Preparation and Proof Rolling section, to determine if a reinforced foundation is needed.

Where a reinforced foundation is required, we recommend over-excavating a minimum of 2 feet. The reinforced zone should extend laterally 2 feet in front of the toe of the fill slope or RSS. A Type II reinforcement grid with a nominal long-term strength of 1,500 lbs/ft should be placed on the bottom of the over-excavation, prior to backfilling. Where encountered, bedrock does not need to be over-excavated and replaced, but should be benched into. The over-excavation should be backfilled using Unclassified Borrow material meeting FP-14 specification 704.06. However, we recommend the Unclassified Borrow material have a maximum particle size of 3 inches.

7.7 Drainage

We observed evidence of pavement subgrade failures during our fieldwork that were likely caused by or exacerbated by subsurface water. Our observations included pavement distress, water within the ditch on the upslope side of the roadway, groundwater seeps within the existing cut and natural slopes above the roadway, piping under the existing roadway, and hydrophilic vegetation within the ditch and slope above the roadway. We recommend incorporating surface and subsurface measures to reduce the potential for water to saturate the pavement subgrade soil, fill slopes, RSS, and to develop behind the soil nail walls.

To reduce the potential for saturation of subgrade soil or buildup of hydrostatic pressure, we recommend installing subsurface drains and underdrains. Subsurface drains and underdrains should consist of 4-inch diameter perforated PVC pipe, encased in gravel, and surrounded by a Class 2, Type E geotextile filter. A solid pipe outlet should be provided at approximately 50 to 100-foot intervals with this type of subsurface drain. We recommend installing:

- subsurface drains in the heel of fill slope keys and at the back of RSS excavations
- underdrains along the edge of the upslope side of the roadway, where surface water can't be controlled, or evidence of groundwater seeps are present. We observed evidence of groundwater in the following station ranges. These ranges should be confirmed during field review and construction.
  - 50+00 to 55+00
  - 92+50 to 97+50
  - 102+50 to 104+00
  - 110+00 to 110+50
  - 114+00 to 124+00

- 128+00 to 130+00
- 132+00 to 135+00
- 137+00 to 143+00

7.8 Shrink and Swell Factors

The soil and rock at the site can be grouped into various material types for earthwork planning purposes. These materials are expected to be encountered during slope excavation and beneath the existing roadway along the alignment.

Exhibit 7-7 presents average shrink and swell factors for the soil and rock types expected to be encountered on the project. These values are based on criteria provided in the FHWA Geotechnical Technical Guidance Manual (U.S. DOT and FHWA, 2007).

Exhibit 7-7: Shrink/Swell Factors for Common Materials

Material	Loose (in situ/bank to truck) Percent Swell[1, 2]	Embankment (in situ/bank to fill) Percent Swell or Shrink[1, 2]
Soil - Gravel (average gradation)	20	-10
Soil - Sand	10	-10
Pumice	65	Not recommended for fill soil
Bedrock – Marble, Quartzite, Rhyolite and Schist	65	45
Bedrock – Shale	80	50
Bedrock -Tuff	50	35

NOTES:

1 A negative value indicates shrinkage.

2 Values rounded to the nearest "5".

Based on the exhibit above and our observations, it is our opinion soil will shrink approximately 5 to 15 percent when excavated and placed as a fill. Similarly, we anticipate bedrock will bulk approximately 40 to 50 percent when excavated and placed as a fill.

7.9 Pavement Design Recommendations

We performed our pavement design calculations in accordance with the 1993 AASHTO Guide for the Design of Pavement Structures, with guidance from the PDDM. We understand the proposed improvements to the upper 2.5 miles will constitute a 4R project and the lower 5.8 miles a 3R project. As such, we performed our analyses considering a 25-year and 20-year design life, respectively.

For the upper 2.5 miles, we provide pavement section design recommendations for HACP over crushed aggregate base (CAB) over the prepared subgrade soil. For the lower 5.8 miles, we provide pavement section design recommendations for HACP over FDR over the prepared subgrade soil. Where a subexcavation is called out in the lower 5.8 miles, the pavement section will consist of HACP over CAB over the subgrade soil (see Subexcavation, Unsuitable Pavement Subgrade section). In areas where the proposed roadway is wider than the existing roadway, the pavement section will consist of HACP over CAB over the subgrade soil. In these locations the FDR and new CAB should be mixed to provide a uniform bearing material.

7.9.1 Traffic Loading

Performance of a pavement system depends on the pavement material and thicknesses, subgrade strength, traffic design vehicles and repetitions, design life, and subgrade drainage characteristics. We understand Reds Meadow Road is a low volume, forest access route that is typically closed from November to May. Based on conversations with Jacobs, we understand the average daily traffic is 445 with a directional distribution of 50 percent. The design vehicles consist of approximately 13.5 percent buses, 26.5 percent passenger cars, and 60 percent pickup trucks and vans. Using these vehicles and repetitions, we estimate 25-year and 20-year design life 18-kip equivalent single-axle loading (ESAL) values of approximately 144,000 and 116,000, respectively.

7.9.2 Design Subgrade Resilient Modulus

We performed six CBR tests to model subgrade soils along the entire 8.3 miles of Reds Meadow Road. The testing resulted in values of 8, 28, 34, 38, 48, and 53 percent, at 95-percent compaction. We converted the average CBR test result of 35 percent to a resilient modulus of approximately 24,900 pounds psi, for use in our analysis.

7.9.3 Full Depth Reclamation

We understand the lower 5.8 miles will be rehabilitated using FDR. The existing AC layer and underlying subgrade soil will be pulverized to a depth of 6 inches and compacted in-place. A layer of HACP will be placed over the compacted FDR layer.

Based on our subsurface explorations, the existing AC thickness ranges from 1.8 to 4.6 inches, with an average of 3 inches, with no underlying base course. The underlying subgrade soil generally consists of poorly graded sand with variable amounts of silt and gravel. We developed our structural coefficient for the FDR considering it will be a mixture of the pulverized AC and subgrade soil.

7.9.4 Recommended Pavement Sections

The following exhibit summarizes design parameters used in our flexible pavement design calculations.

Exhibit 7-8: Flexible Pavement Design Parameters

Table Title		Table Title
Pavement Design Life (years)		25 and 20
Subgrade Resilient Modulus, MR (psi)		24,900
Reliability, R (%)		75
Standard Normal Deviate, ZR		-0.674
Serviceability Loss, ΔPSI (P0 of 4.2 to Pt of 2.0)		2.2
Overall Deviation, So		0.49
Structural Layer Coefficient	HACP	0.42
	CAB	0.13
	FDR	0.11
Drainage Coefficient	CAB	1.0
	FDR	1.0

NOTES:
psi = pounds per square inch
% = percent

Using the above parameters, we summarize our pavement section alternatives in Exhibit 7-9.

Exhibit 7-9: Flexible Pavement Section Alternatives

HACP	CAB	FDR
3.0	4.0	-
3.0	-	6.0

NOTES:
Our calculations resulted in the same pavement sections for 20-year and 25-year design life.

8 CONSTRUCTION CONSIDERATIONS

The applicability of the design recommendations provided in this report is contingent on good construction practice and adherence to the project specifications. Poor construction techniques and methods may alter conditions from those on which our recommendations are based and result in poor performance. Our analyses assume that this project will be constructed according to FP-14. The following sections provide additional construction considerations for this project.

8.1 Site Preparation

We recommend that brush and other vegetation be cleared, and roots and stumps be removed from all areas to be graded. All surface and subsurface structures associated with current development of the site, including utility poles, fence poles, underground utilities and other deleterious material, should also be removed. Any existing surficial topsoil, soil containing organics, and pumice-rich soil should be stripped and removed from areas proposed for new fill, pavement, or structural improvements. The depth of this removal is anticipated to vary. Topsoil, organic-rich soils, and pumice-rich soils are not considered suitable for reuse as fill and should be removed from the site or stockpiled for reuse in landscaping areas.

8.2 Earthwork and Grading

Earthwork, including placement of fill and subgrade preparation, should conform to the requirements provided in the FP-14 and the recommendations provided in the following sections.

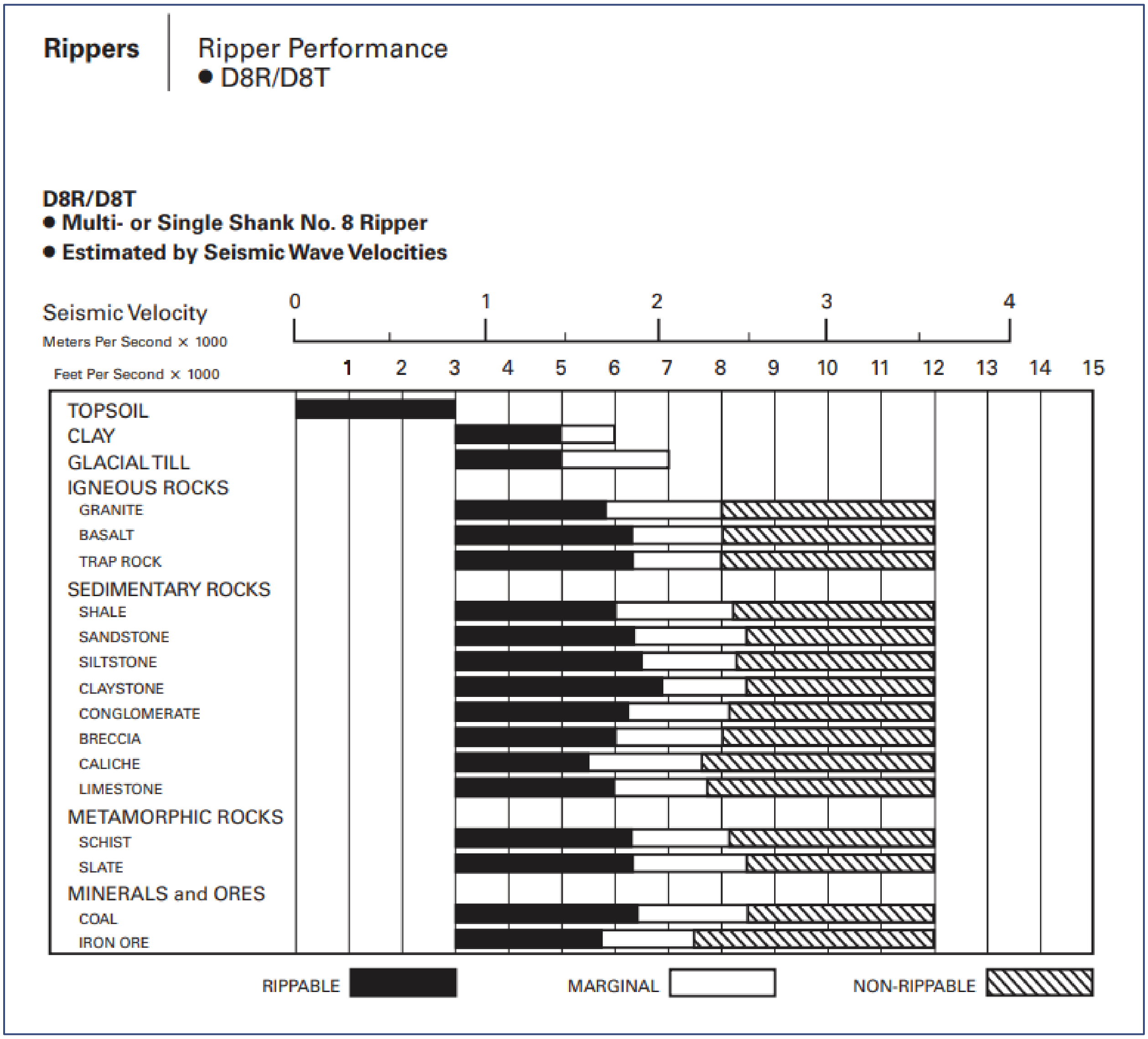
8.2.1 Excavation

We anticipate the surficial soil units (af and Qts) can be excavated using conventional excavating equipment, such as a rubber-tired backhoe or tracked hydraulic excavator. We anticipate boulder-sized clasts of bedrock will be encountered within the soil materials during construction. Considering the hardness of the boulders observed at the site, reducing boulders to be used as fill may be difficult.

Excavation in bedrock materials, including constructing cut slopes, performing removals to install RSS reinforcement, or drilling for soil nails may also be difficult. The bedrock we observed at the site is Jurassic to Triassic in age and generally medium strong to very strong. We performed a geophysical study to evaluate the rippability of the bedrock. Geovision Geophysical Services, Inc. (Geovision) performed 15 seismic refraction lines along the alignment from October 15 to 20, 2018. The Geovision report based their interpretation of the rock rippability based on a schist rock type, which aligned the closest with the rock types observed along the alignment. This rock type is assumed to be rippable by a Caterpillar D8 Ripper to a P-wave velocity of 6,250 feet per second (ft/s) and marginally rippable up to 8,250 ft/s if the rock is sufficiently jointed and fractured. Exhibit 8-1 below presents the D8 Ripper's rippability characteristics versus seismic wave velocity for different rock types, presented within the Caterpillar Performance Handbook 48

(Caterpillar, 2018). Exhibit 8-2 on the next page summarizes the rippability based on the Geovision report.

Exhibit 8-1: D8R Ripper Performance and Seismic Wave Velocity



NOTES:

1. Rock rippability recommendations for the alignment are based on schist rock type.

Exhibit 8-2: Summary of Rippability Velocities and Estimated Depths[1]

Seismic Line Number	Project Feature	Top of Rippable Rock Depth[2,5]	Top of Marginally Rippable Rock Depth[3,5]	Top of Nonrippable Rock Depth[4,5]
S-1	RW-1 / bedrock rippability	Near surface along W and C portion. 7 to 10 ft in E portion.	23 to 35 ft in W and C portion. Up to 50 ft possible in E portion.	>40 ft in W portion. >50 ft in C and E portion.
S-2	RW-2A / depth to bedrock	14 to 17 ft S portion 12 ft C portion 10 ft N portion	35 ft N portion	38 ft N portion
S-3	RW-2A / depth to bedrock	7 to 9 ft S portion 3 to 6 ft C portion 1 to 2 ft N portion	>30 ft S portion >25 ft C portion >10 ft N portion	>30 ft S portion >25 ft C portion >10 ft N portion
S-4	RW-2B / bedrock rippability	5 to 12 ft S portion 8 to 10 ft C and N portions	8 to 16 ft S portion 10 to 12 ft C portion 10 to 17 ft N portion	11 to > 30 ft S portion 11 to > 30 ft C portion 30 ft N portion
S-5	RW-2B / depth to bedrock	9 to 15 ft SW portion 15 ft C and NE portion		
S-6	Depth to bedrock	16 to 18 ft W portion 12 to 14 ft C and E portions		
S-7	Depth to bedrock		8 to 10 ft SW and C portions 10 to 13 ft NE portion	
S-8	RW-4 / bedrock rippability	1 to 5 ft S portion 6 to 10 ft C and N portions		20 ft N portion
S-9	RW-4 / depth to bedrock	7 to 17 ft W portion 20 to 25 ft C portion 15 to 20 ft E portion		
S-10	Depth to bedrock	19 to 21 ft NW portion 20 to 22 ft C portion 13 to 18 ft E portion		
S-11	Depth to bedrock	8 to 10 ft	Below about 10 ft	
S-12	RW-8 / bedrock rippability			3 to 6 ft NW portion 3 ft C portion 3 to 8 ft SE portion
S-13	RW-2A / bedrock rippability			7 to 10 ft SE portion 3 to 6 ft C portion 7 to 9 ft NW portion
S-14	Depth to bedrock	28 ft W portion 22 to 25 ft C portion 12 to 15 ft E portion		
S-15	Depth to bedrock	6 to 7 ft S and C portion 5 ft N portion		

NOTES:

1 Rippability is based on a Caterpillar D8 Ripper. The contractor should determine if this type of equipment is suitable for the proposed construction.

2 Rippable rock is assumed to be rippable below a velocity of 6,250 ft/s.

3 Marginally rippable rock is assumed to have a velocity up to 8,250 ft/s.

4 Nonrippable rock is assumed to have a velocity over 8,250 ft/s.

5 Seismic lines are described as Northern (N), Southern (S), Central (C), Eastern (E), Western (W), or variations of the Cardinal directions.

In areas where the existing cut slope does not expose a continuous rock mass, excavation may be able to be accomplished using mechanical methods of ripping. We anticipate that the excavation into bedrock at the site may be feasible using conventional equipment at RW-1 and in some areas of RW-2B, but drilling and blasting will likely be required at the other rock cut slopes due to the strength of the bedrock. Blasting methods should consist of controlled blasting to form permanent cut slopes.

8.2.2 Temporary Excavations

Temporary excavations are the responsibility of the contractor. The appropriate backcut gradient will depend on the conditions exposed during construction and the contractor's means and methods.

8.2.3 Subgrade Preparation and Proof Rolling

Proper subgrade preparation is required for embankment, pavement, and RSS performance. Foundation subgrade excavations should be prepared in accordance with FP-14 204.09. The subgrade should be cleaned of all undocumented fill, debris, or loose/soft material (including pumice-rich soil). After the subgrade is excavated to the required depth, the subgrade should be proof-rolled with a fully loaded, tandem-axle, 10-yard dump truck or equivalent. If proof rolling cannot be performed, a T-probe should be used to check the firmness of the ground. Areas that are identified as being loose, soft, or yielding during proof-rolling or probing should be compacted in place, removed and reconditioned, or replaced with fill material as discussed in the Fill Placement and Compaction section of this report. Care should be taken during proof-rolling and subgrade preparation to avoid disturbing subgrade soils and supporting soils that will remain in place, as they can rut and pump under repeated construction traffic. The final subgrade surface should always be sloped to promote positive drainage and kept free of water. Leaving the subgrade elevation high until final grading begins is a means to reduce the potential for disturbance to the final subgrade materials.

8.2.4 Fill Placement and Compaction

Fills and backfill should meet the requirements of Section 704 of FP-14. All fill material placed should consist of soil that is free of organics, contaminants, debris, pumice-rich soil, and rock fragments larger than 3 inches. In addition, fill material should be placed in horizontal lifts, with the loose lift thickness not to exceed 8 inches for heavy equipment compactors and 4 inches for hand-operated compactors and be compacted to a dense and unyielding condition. Thinner lifts may be required, depending on the contractor's equipment.

Backfill compaction requirements should be in accordance with Section 204.11 of FP-14. Based on our laboratory testing and for preliminary planning purposes, we anticipate that compaction requirements would follow FP-14 204.11(c), but this should be confirmed during construction.

8.3 Rock Scaling

The contractor should perform rock scaling where rock is exposed in the existing slopes that are to remain and new rock cut slopes adjacent to the inboard side of the roadway. Scaling should be performed using hand methods or limited mechanical equipment and be observed/directed by the field engineer. Scaling should remove loose rock on the face of the slope up to the brow of the slope.

In Appendix F we provide photographs of the existing rock slopes that are too remain. In Exhibit 8-3 we have summarized the approximate limits of these existing rock slopes.

Exhibit 8-3: Scaling Existing Slopes to Remain

Stations	
Begin	End
11+80	13+00
14+50	15+15
72+80	74+00

8.4 Culverts

We understand culverts have an average service life expectancy of 50 years. Based on the corrosion potential of the onsite soil, we have estimated the average service life will be less than 50 years.

Section 7.3.6.3.4 of the PDDM provides guidance with respect to steel pipe mitigation for corrosion potential. To achieve an average service life of 50 years in accordance with the Exhibit 7.3B of the PDDM, we provide the following alternatives:

- 12 gage pipe thickness
- aluminum coated steel
- bituminous coating
- concrete lining

- polymer coating

8.5 Soil Nail Walls

8.5.1 Construction

The typical construction sequence of a soil nail wall consists of excavating to a specified level such that nail holes can be drilled from a platform. Once the holes are drilled, steel bars, i.e., soil nails, are placed in the holes and the holes are filled with cement grout. A temporary shotcrete facing system is then constructed to support the exposed face before the subsequent level is excavated. After the bottom of the excavation is reached and the nails and temporary facing are installed for each level, the final, permanent facing is constructed.

The contractor should select appropriate means and methods for stabilizing the exposed excavation face, controlling "overbreak", drilling and completing the grouted soil nails, and constructing the proposed soil nail wall as intended on the design plans and specifications. The contractor should provide a submittal with his means and methods of construction for review prior to the start of construction.

We anticipate the temporary cut to construct the soil nail wall and drilling to install the soil nails will encounter talus and slopewash in some locations and bedrock in other areas. We observed the talus and slope wash exposed in existing slopes in areas of pumice-rich soil to be loose and prone to raveling and sloughing. We anticipate the excavation in these areas will be prone to raveling and sloughing during construction and the soil nail drilled holes will be prone to caving. At the same time, the talus and slopewash contains clasts of bedrock that vary in size up to boulders. Where boulders or bedrock is encountered, hard drilling conditions should be anticipated.

Where multiple rows of soil nails are specified, we anticipate the contractor will use a construction bench to facilitate installation of the upper row of nails and temporary shotcrete facing. The excavated bench may not be able to support the contractor's equipment without some compaction effort to densify the soil or importing soil to create the necessary width. The contractor should confirm intermediate stages of construction will maintain an adequate factor of safety.

Where backfilling is required behind an extension of the wall facing at the top, light mechanical tampers should be used within 3 feet of the wall extension.

8.5.2 Testing

Verification and proof testing should be performed as a part of soil nail wall construction. These tests should be performed on the soil nails in accordance with the procedures and acceptance criteria in GEC No. 7 (Lazarte and others, 2015).

We recommend performing a verification test for each 500 square feet of shotcrete wall. The verification testing should be performed on non-production, sacrificial nails prior to construction to confirm the contractor's means and methods are able to achieve the nominal pullout resistance values used in design. The tests should be performed using the same construction methods that will be used on production nails.

Exhibit 8-4: Number of Verification Tests Recommended

Wall	Number of Verification Tests
27	3
34	4
48	1
50	1
62	7
85	1
92 & 93	1
95	1
99	1
100	2
111	1
119	4
131	1

At least 5-percent of the production soil nails should be proof tested. The locations of the proof tests should be determined by the CFL construction manager based on the conditions observed during installation. The proof tests should be dispersed throughout each wall.

8.6 Pavement Materials

The following exhibit summarizes our recommendations for pavement material selection.

Exhibit 8-5: Recommended Paving Materials

Material	FP-14 Specification	Additional Requirements/Comments
CAB	Section 301	• Gradation C, D, or E (FP-14 703.05) • CBR-value > 60
HACP	Section 401	• Mix design shall meet requirements for ESAL values less than 300,000
Binder	-	• PG 64-28M

NOTES:
ESAL = Equivalent Single-Axle Loading
PG = Performance Grade

Our recommended pavement thicknesses require the use of the specific FP-14 specification identified in Exhibit 8-5. If an alternate base course or HACP specification from FP-14 is selected, pavement sections may need to be redesigned.

9 CONSTRUCTION OBSERVATION

Geotechnical design recommendations are developed from a limited number of explorations and tests. Therefore, recommendations may need to be adjusted in the field. To this end, we recommend that a construction observation and monitoring program be implemented for the project and that Shannon & Wilson be retained to monitor the geotechnical aspects of construction, particularly excavations, installation of culverts, subgrade preparation, fill placement, construction of RSS, soil nail walls, scaling, and rock slope construction. This monitoring would allow us to confirm that conditions encountered are consistent with those extrapolated between the explorations and provide recommendations should conditions be revealed during construction that are different from those anticipated.

10 LIMITATIONS

The analyses, conclusions, and recommendations presented in this report are based on our understanding of the project and a limited number of subsurface explorations and laboratory test results. We assume that these explorations and tests are representative of the subsurface conditions beneath the site; that is, the subsurface conditions everywhere are not significantly different from those disclosed by the explorations and tests.

This report was prepared for the exclusive use of Jacobs and CFL for the use in design of the Reds Meadow Road project. It should be made available to prospective contractors and/or the contractor for information on factual data only, and not as warranty of subsurface conditions, such as those interpreted from the exploration logs and presented in the discussion of subsurface conditions included in this report.

Unanticipated soil conditions are commonly encountered and cannot be fully determined by a limited exploration and testing program. Such unexpected conditions frequently require that additional expenditures be made to attain a properly constructed project. Therefore, some contingency fund is recommended to accommodate such potential extra costs.

Within the limitations of scope, schedule and budget, the analyses, conclusions, and recommendations presented in this report were prepared in accordance with generally accepted professional geotechnical and geological principles and practice in this area at the time this report was prepared. We make no other warranty, either express or implied.

The scope of our services did not include an evaluation regarding the presence or absence of hazardous or toxic materials in the soil, surface water, groundwater, or air, on or below or around this site. If such contamination exists, it would not be possible to determine it within this limited scope of work.

Shannon & Wilson has prepared the document, "Important Information About Your Geotechnical/Environmental Report," to assist you and others in understanding the use and limitations of our reports.

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