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1.0 INTRODUCTION

1.1 Project Description

This section describes the proposed action and the project alternatives developed to meet the purpose and need described below, while avoiding or minimizing environmental impacts. There is one Build Alternative and the No-Build Alternative.

The California Department of Transportation (Caltrans), in cooperation with the Santa Cruz County Regional Transportation Commission (SCCRTC) and the County of Santa Cruz, proposes to widen State Route (SR) 1 to include auxiliary lanes, accommodate bus-on-shoulder (BOS) operations between the Freedom Boulevard and State Park Drive interchanges, and construct Coastal Rail Trail Segment 12.

One build alternative and the no-build are proposed for further consideration. The project is located in Santa Cruz County on SR 1 from post mile (PM) 8.1, south of Freedom Boulevard, to PM 10.7, north of State Park Drive. The total length of the project on SR 1 is 2.6 miles. Within the limits of the proposed project, SR 1 is a controlled access freeway with two 12-foot lanes; shoulder width varies within project limits. The average width of the inside shoulders is approximately 5 feet, and the average width of the outside shoulders is approximately 10 feet. The project also includes the proposed Coastal Rail Trail Segment 12, which would extend approximately 1.14 miles along the SCCRTC-owned Santa Cruz Branch Line railroad right-of-way, between Rio Del Mar Boulevard and State Park Drive. Approximate project location is shown in Plate No. 1, Project Vicinity Map.

Coordinates shown are based on CCS 1983 (2011 Epoch 2017.5, Zone 3) and Orthometric Heights shown are NAVD 88 based on Santa Cruz County Monuments control provided by Caltrans District 5. Coordinates and elevations are in U.S. Survey feet. It is our understanding that any boring drilled before 1991 used NGVD29 datum for registering elevations. These elevation values from as-built borings were approximately converted to NAVD88 datum by increasing the elevation values by 2.72 feet. Elevations based on NGVD29 datum were converted into NAVD88 datum date by adding 2.72 feet (Elev. NAVD88 = Elev. NGVD29 + 2.72 feet).
1.1.1 Project Background

SR 1 is a primary route connecting the southern and central areas of Santa Cruz County and is the only continuous commuter route linking Watsonville, Capitola, Aptos, Cabrillo College, Santa Cruz, and the University of California Santa Cruz. SR 1 is also a southern terminus for SR 9 and SR 17, which bring heavy tourist traffic to coastal destinations in Santa Cruz and Monterey counties.

Improvements in the project area were addressed previously in a combined Tier I/ Tier II EIR with a Finding of No Significant Impact (FONSI), which was adopted in December 2018. The Tier I component, referred to as the corridor improvement project, proposed approximately 8.9 miles of new high-occupancy vehicle (HOV) lanes, HOV on-ramp bypass lanes, auxiliary lanes, pedestrian and bicycle overcrossings, and reconstructed interchanges. It was recognized that the Tier I project would likely be implemented in phases.

The proposed project is the third phase of the improvements described in the Tier I EIR/FONSI. Construction of the proposed project would allow for future outside highway widening to accommodate the future Tier I HOV lanes.

**Purpose**

The purpose and objectives of the project are listed below.

- Reduce congestion along SR 1 through the project limits.
- Enhance bicycle and pedestrian connectivity along Segment 12 of the Coastal Rail Trail.
- Promote the use of alternative transportation modes to increase transportation system capacity and reliability.
- Provide Coastal Rail Trail access across SR 1 at the two railroad bridges.

**Need**

The project is needed for the following reasons.

- Several bottlenecks along SR 1 in the southbound and northbound directions cause congestion during peak hours, significantly delaying drivers.
• Cut-through traffic, or traffic on local streets, is increasing because drivers are seeking to avoid congestion on SR 1.

• There are limited opportunities for pedestrians and bicyclists to safely cross SR 1 and navigate the project corridor, even though portions of the project area are designated as regional bicycle routes.

• There are insufficient incentives to increase transit service in the SR 1 corridor because congestion threatens reliability and cost-effective transit service delivery.

1.1.2 Project Alternatives

There is one Build alternative and one No Build alternative being considered for this project.

1.1.3 Build Alternative

Auxiliary Lanes

Auxiliary lanes are designed to improve merging operations and reduce conflicts between traffic entering and exiting SR 1 by connecting the on-ramp of one interchange to the off-ramp of the next; they are not designed to serve through traffic. A southbound auxiliary lane and a northbound auxiliary lane would be added to the following segments of SR 1.

• Between the Freedom Boulevard and Rio Del Mar Boulevard interchanges.

• Between the Rio Del Mar Boulevard and State Park Drive interchanges.

The total roadway widening would be approximately 2.6 miles in length. Southbound, the auxiliary lane would begin at the existing State Park Drive loop on-ramp and end at the existing off-ramp to Freedom Boulevard. Northbound, the auxiliary lane would begin at the existing Freedom Boulevard on-ramp and end at the existing diagonal off-ramp to State Park Drive.

The new auxiliary lanes would be 12 feet wide. From Freedom Boulevard to Rio Del Mar Boulevard, the width needed for the new lane would be added in the median. The existing median barrier would be reconstructed in its current location. From Rio Del Mar Boulevard to State Park Drive, the width needed for the new lane would be added outside the existing shoulders; the outside shoulders would be standard 10-foot-wide.
Moosehead Drive to the south of SR 1, south of Aptos Creek, would be realigned where it runs parallel to SR 1 due to the outside widening of SR 1. A new retaining wall would be placed along the outside freeway shoulder to support the realignment that would include horizontal and vertical adjustments.

**Structures – State Route 1**

The Build Alternative would include the replacement of the two Santa Cruz Branch Line railroad bridges over SR 1 and widening of the SR 1 bridge over Aptos Creek and Spreckels Drive to accommodate the proposed auxiliary lanes. The existing Santa Cruz Branch Line railroad bridges (overcrossing structures) are proposed to be replaced with longer spans to accommodate the planned SR 1 ultimate improvements that are a six-through-lane concept plus an auxiliary lane in each direction between interchanges, approved in the *Final Environmental Impact Report/Environmental Assessment with a Finding of No Significant Impact for the Tier I High Occupancy Vehicle (HOV) Lanes and Tier II 41st Avenue to Soquel Avenue/Drive Auxiliary Lanes Project* (Tier I/Tier II Final EA/EIR/FONSI).

In addition to the proposed railroad bridges, new trail overcrossings would be constructed adjacent to the new railroad bridges for Coastal Rail Trail Segment 12 for the SR 1 ultimate improvements.

The widening of the SR 1 bridge over Aptos Creek and Spreckels Drive would occur on the south side of SR 1 only and require abutment walls along the existing embankments along the south side of Aptos Creek and the embankment on the north side of Spreckels Drive. The widened bridge would accommodate six lanes, each 12-feet wide (four through-lanes plus an auxiliary lane in each direction), 10-foot-wide outside shoulders, and a 9-foot-wide median with a 5-foot wide inside shoulder for southbound SR 1 and a 2-foot-wide inside shoulder for northbound SR 1. To accommodate the future SR 1 ultimate improvements of six through-lanes plus an auxiliary lane in each direction, the SR 1 bridge over Aptos Creek and Spreckels Drive would be widened to the north (inland) side as part of a future project.

**Retaining Walls – State Route 1**

The build alternative would include retaining walls at the following locations along SR 1. Retaining wall details are shown in the table below and in APS plans attached in Appendix VII.
TABLE 1: RETAINING WALL INFORMATION

<table>
<thead>
<tr>
<th>Wall No.</th>
<th>Wall Type</th>
<th>Station Begin</th>
<th>Station End</th>
<th>Approx. Length</th>
<th>Maximum Design Height</th>
</tr>
</thead>
<tbody>
<tr>
<td>Retaining Wall No.1</td>
<td>SHGA</td>
<td>“SR1” Line Sta. 258+55.63</td>
<td>“SR1” Line Sta. 263+00.63</td>
<td>445'-0&quot;</td>
<td>22'-4&quot;</td>
</tr>
<tr>
<td>Retaining Wall No.2</td>
<td>SHGA</td>
<td>“SR1” Line Sta. 258+90.63</td>
<td>“SR1” Line Sta. 261+25.63</td>
<td>235'-0&quot;</td>
<td>22'-4&quot;</td>
</tr>
<tr>
<td>Retaining Wall No.3</td>
<td>SHGA</td>
<td>“SR1” Line Sta. 265+55.62</td>
<td>“SR1” Line Sta. 268+45.62</td>
<td>290'-0&quot;</td>
<td>18'-3&quot;</td>
</tr>
<tr>
<td>Retaining Wall No.4</td>
<td>SHGA</td>
<td>“SR1” Line Sta. 269+71.01</td>
<td>“SR1” Line Sta. 270+69.41</td>
<td>100'-0&quot;</td>
<td>8'-6&quot;</td>
</tr>
<tr>
<td>Retaining Wall No.5</td>
<td>Type 7B</td>
<td>“MH” Line Sta. 10+49.11</td>
<td>“SR1” Line Sta. 277+01.63</td>
<td>395'-0&quot;</td>
<td>20'</td>
</tr>
<tr>
<td>Retaining Wall No.6</td>
<td>MSE</td>
<td>“SR1” Line Sta. 277+01.94</td>
<td>“SR1” Line Sta. 278+92.59</td>
<td>190'-8&quot;</td>
<td>20'</td>
</tr>
<tr>
<td>Retaining Wall No.7</td>
<td>MSE</td>
<td>“SR1” Line Sta. 281+58.09</td>
<td>“SR1” Line Sta. 284+40.89</td>
<td>282'-9.5&quot;</td>
<td>22'</td>
</tr>
<tr>
<td>Retaining Wall No.8</td>
<td>Soil Nail &amp; SHGA</td>
<td>“SR1” Line Sta. 284+40.89</td>
<td>“SR1” Line Sta. 292+79.58</td>
<td>694'-0&quot;</td>
<td>26'-6&quot;</td>
</tr>
<tr>
<td>Retaining Wall No.9</td>
<td>Soil Nail &amp; SHGA</td>
<td>“SR1” Line Sta. 289+00.00</td>
<td>“SR1” Line Sta. 295+29.39</td>
<td>610'-8&quot;</td>
<td>20' 6&quot;</td>
</tr>
</tbody>
</table>

Notes:
1. SHGA: Sub-Horizontal Ground Anchor
2. MSE: Mechanically Stabilized Earth Retaining Wall

**Sound Walls**

The build alternative would include sound walls along Route 1. Sound wall details are shown in the table below.

TABLE 2: SOUND WALL INFORMATION

<table>
<thead>
<tr>
<th>Sound Wall No.</th>
<th>Location</th>
<th>Beginning Station (“SR1”)</th>
<th>End Station (“SR1”)</th>
<th>Approx. Length</th>
<th>Maximum Design Height</th>
</tr>
</thead>
<tbody>
<tr>
<td>S68</td>
<td>Northbound</td>
<td>192+46.58</td>
<td>224+47.78</td>
<td>3,293</td>
<td>16</td>
</tr>
<tr>
<td>S71</td>
<td>Southbound</td>
<td>201+16.95</td>
<td>233+40.57</td>
<td>3,280</td>
<td>16</td>
</tr>
<tr>
<td>S86a</td>
<td>Northbound</td>
<td>261+80.90</td>
<td>267+48.85</td>
<td>606</td>
<td>16</td>
</tr>
<tr>
<td>S87</td>
<td>Southbound</td>
<td>267+35.67</td>
<td>277+03.03</td>
<td>1,057</td>
<td>16</td>
</tr>
<tr>
<td>S89</td>
<td>Southbound</td>
<td>276+62.13</td>
<td>284+40.89</td>
<td>885</td>
<td>16</td>
</tr>
<tr>
<td>S90</td>
<td>Northbound</td>
<td>268+00.00</td>
<td>287+00.62</td>
<td>1,862</td>
<td>16</td>
</tr>
<tr>
<td>S93</td>
<td>Southbound</td>
<td>285+73.04</td>
<td>291+51.74</td>
<td>585</td>
<td>16</td>
</tr>
<tr>
<td>SB-1</td>
<td>Southbound</td>
<td>259+81.73</td>
<td>261+13.65</td>
<td>141</td>
<td>16</td>
</tr>
</tbody>
</table>

**Bus on Shoulder Features (BOS)**

BOS features are proposed, which would allow future bus operations on the outside shoulders of SR 1 through the interchanges during peak congestion periods. At the Freedom Boulevard, Rio Del Mar Boulevard, and State Park Drive interchanges, the project would widen and improve SR 1 shoulders, which currently lack the width and pavement structural section to support bus operations.
Cross Section – Route 1 Bus on Shoulders

The added auxiliary lanes coupled with the BOS improvements allow the transit operator to use the auxiliary lane in between interchanges and use the outside shoulder between the off-ramp and on-ramps through the interchanges. Within the Freedom Boulevard, Rio Del Mar Boulevard, and State Park Drive interchange areas, the highway shoulders would be 12 feet wide.

Other Features – Route 1 Bus on Shoulders

New overhead and roadside signs would be installed to advise motorists that only buses are allowed to use the highway shoulders through interchanges during peak traffic hours. Along northbound SR 1, a sign would be provided south of each of the three interchanges in the project area. Along southbound SR 1, a sign would be installed north of each interchange. Shoulders would be painted red to indicate bus-only use.

Coastal Rail Segment 12

Within the project area, the existing railroad right-of-way is generally in the range of 40 to 55 feet wide, with the existing railroad tracks generally in the center of the right-of-way. The existing railroad has at-grade crossings at State Park Drive, Aptos Creek Road, Parade Street and Trout Gulch Road, with bridges over SR 1 at two locations, Soquel Drive, Aptos Creek and Valencia Creek, and crosses under Rio Del Mar Boulevard. The proposed Coastal Rail Trail Segment 12 includes the construction of a paved bicycle and pedestrian shared use trail within the Santa Cruz Branch Rail Line right-of-way on the inland side of the tracks consistent with the approved Monterey Bay Sanctuary Scenic Trail (MBSST) Network Master Plan (MBSST Network Master Plan), with the option of being implemented in two phases. Phase 1 improvements would be an interim condition and Phase 2 improvements would be the ultimate Segment 12 improvements described above. The limits of Coastal Rail Trail Segment 12 extend from the southern terminus of the trail segment at Sumner Avenue just to the south of the Rio Del Mar Boulevard underpass to the northern terminus at State Park Drive. The trail segment would include an at-grade crossing of State Park Drive and an at-grade trail connection to Sumner Avenue just south of the Rio Del Mar Boulevard underpass where the existing railroad tracks pass under Rio Del Mar Boulevard.
Retaining Walls

Retaining walls would be constructed in the following locations for the Coastal Rail Trail Segment 12 alignment.

Phase 1 Interim Improvements

- Just west of Soquel Drive - An approximate 5-foot high, 60-foot long retaining wall on the south side of the trail.
- Just east of Aptos Creek —An approximate 18-foot high, 140-foot long retaining wall on the south side of the trail and an approximate 6-foot high, 140-foot long retaining wall on the inland side of the trail.

Phase 2 Ultimate Improvements

- North of SR 1 (towards State Park Drive) – An approximate 6-foot high, 300-foot long retaining wall on the inland side of the trail.
- SR 1 to Soquel Drive—Retaining wall varying in height between approximately 5-feet and 20-feet, approximately 300-feet long on the inland side of the trail.
- Aptos Creek to Aptos Creek Road—Retaining wall varying in height between approximately 2-feet and 18-feet, approximately 400-feet long on the inland side of the trail.
- Trout Gulch Road to Valencia Creek—Retaining wall varying in height between approximately 2-feet and 18-feet, approximately 450-feet long on the inland side of the trail.
- South of SR 1 (towards Rio Del Mar Boulevard)—An approximate 12-foot-high, 400-foot long retaining wall on the inland side of the trail.
- Under Rio Del Mar Boulevard - Retaining wall varying in height between approximately 4-feet and 16-feet, approximately 1000-feet long on the inland side of the trail.

1.1.4 No Build Alternative

Under the No-Build Alternative, there would be no construction of auxiliary lanes or BOS features on SR 1 within the project area, and Coastal Rail Trail Segment 12 would not be
constructed. The existing transportation facilities within the project area would remain unchanged. The No-Build Alternative assumes the construction of other planned and programmed projects in the region, including other auxiliary lanes projects on SR 1 and other segments of the Coastal Rail Trail.

1.2 Exception to Policy

No exception to Caltrans policy is required.

2.0 GEOTECHNICAL INVESTIGATION

Site reconnaissance and field investigations were not performed at this preliminary stage since it was not part of the scope. Site reconnaissance and field investigation will be performed during PS&E phase.

2.1 Existing As-Builts Plans

The following geotechnical reports and As-Built LOTBs were referred:

1) As-built Log of Test Boring, State Park Drive Overcrossing, Bridge No. (36-28), dated May 16, 1961. Three (3) penetration tests (depth between 20.7 and 31.2 feet) and two (2) borings (depth between 26.9 and 40.5 feet) were included in the as-built LOTB.

2) As-built Log of Test Boring, Retaining Wall at Proposed Interchange at Freedom Blvd, WO. 133851, CU. 04220, dated August 15, 1967. Three (3) penetration tests (depth between 42 and 52.1 feet) and one (1) boring (70 feet depth) were included in the as-built LOTB.

3) As-built Log of Test Boring, Rob Roy Junction Overcrossing, Bridge No. (36-22), dated November 6, 1969. Two (2) penetration tests (depth between 40 and 58 feet) and one (1) boring (40.4 feet depth) were included in the as-built LOTB.

4) As-built Log of Test Boring, Rio Del Mar Boulevard OC, Bridge No. 36-23, dated February 14, 1968. Two (2) penetration tests (depth between 8.8 and 11.5 feet) and one (1) boring (26 feet depth) were included in the as-built LOTB.

Copies of the reference As-Built LOTBs are included in Appendix III.
2.2 Technical Guidelines and Published Maps

The following technical guidelines and published maps were referred:


3.0 GEOTECHNICAL CONDITIONS

3.1 Geology

The project lies on the coastal plain between the Santa Cruz Mountains and north shore of Monterey Bay being contained within the geologically complex and seismically active California Coast Ranges Geomorphic Province. Sub-parallel northwest-trending faults, mountain ranges, and valleys characterize Coast Ranges topography.

The Jurassic-Cretaceous Franciscan Complex and Great Valley sequence sediments comprise the oldest Coast Ranges bedrock units. Subsequently, younger volcanic and sedimentary rocks were deposited throughout the province. Extensive late Cretaceous through early Tertiary folding and thrust faulting created complex geologic structural conditions that underlie the highly varied topography of today. Valley bedrock of the Coast Ranges is covered by thick locally and distally derived alluvium and soils.

Geologic descriptions and distribution of units in the broader project area are drawn from USGS OF97-489, Geologic Map of Santa Cruz County, and shown on Plate No. II-1. Actual distribution of geological units may vary from mapped distributions.

Bedrock of the project area consists primarily of the Purisima Formation (Tp; Pliocene and upper Miocene) described as: very thick bedded yellowish-gray tuffaceous and
diatomaceous siltstone containing thick interbeds of bluish-gray, semi-friable, fine-grained andesitic sandstone, with a thickness of 3,000 feet (Brabb, 1989). The unit is mapped as outcropping between about SCR-1-PM 10.15 to 9.99 and SCR-1-PM 9.67 to 9.59. over about 20% of the project alignment where drainage has cut down through Quaternary units.

Quaternary units of relevance to the project include:

- **Qb – Basin deposits (Holocene)** - Unconsolidated, plastic, silty clay and clay rich in organic material. Locally contain interbedded thin layers of silt and silty sand. Thickness highly variable; may be as much as 90 feet thick underlying some sloughs. This unit is mapped to underlie the Project from about SCR-1-PM 9.01 until past the southern Project extent.

- **Qal - Alluvial deposits, undifferentiated (Holocene)** - Unconsolidated, heterogeneous, moderately sorted silt and sand containing discontinuous lenses of clay and silty clay. Locally includes large amounts of gravel. Thickness is highly variable. The unit underlies the Project between SCR-1-PM 9.59 to 9.84 and presumably overlies unit Tp.

- **Qof - Older flood-plain deposits (Holocene)** — Unconsolidated, fine-grained sand, silt, and clay. Lower parts of these fluvial aggradational deposits include large amounts of gravel. The unit appears to overlie unit Tp and is mapped to underlie the Project between about SCR-1-PM 9.84 to 9.67 and between SCR-1-PM 9.59 to 9.01.

- **Qcl - Lowest emergent coastal terrace deposits (Pleistocene)** — Semi-consolidated, generally well-sorted sand with a few thin, relatively continuous layers of gravel. Deposited in nearshore high-energy marine environment. Thickness variable; maximum approximately 40 ft. Weathered zone ranges from 5 to 20 ft thick. As mapped, locally includes many small areas of fluvial and colluvial silt, sand, and gravel, especially at or near old wave-cut cliffs. The unit is mapped to underlie the project from about SCR-1-PM 10.15 north west to past the end of the Project extent.

Quaternary units adjacent to the project alignment include:
• Qt - Terrace deposits, undifferentiated (Pleistocene)—Weakly consolidated to semi-consolidated heterogeneous deposits of moderately to poorly sorted silt, silty clay, sand, and gravel. Mostly deposited in a fluvial environment. Thickness highly variable; locally as much as 60 ft thick. Some deposits are relatively well indurated in upper 10 ft of weathered zone. The unit is not mapped to outcrop in the project area;

• Qcu- Coastal terrace deposits, undifferentiated (Pleistocene)—Semi-consolidated, moderately well sorted marine sand with thin, discontinuous gravel-rich layers. May be overlain by poorly sorted fluvial and colluvial silt, sand, and gravel. Thickness variable; generally less than 20 ft thick. May be relatively well indurated in upper part of weathered zone. The unit is a distinctive geological feature of the coastal areas of Santa Cruz, however it is not mapped to outcrop within the project alignment;

• Qar- Aromas Sand (Pleistocene) - heterogeneous sequence of mainly eolian and fluvial sand, silt, clay, and gravel. Total thickness may be more than 800ft;

• Qae- Aromas Sand, Eolian lithofacies (Pleistocene) - Moderately well sorted eolian sand. Highly variable degree of consolidation owing to differential weathering. May be as much as 200 ft thick without intervening fluvial deposits. 10 to 20 ft of each dune sequence is oxidized and relatively indurated. Lower part of each dune sequence below weathering zone may be essentially unconsolidated.

3.2 Topsoil – Soil Survey Review

The topsoil survey review data was derived from the USDA Natural Resources Conservation Service, 2019, “Soil Survey Geographic Database for Santa Cruz County, California, ca087”. The proposed Project covers several soil types and are shown in Plate 3. Soil data and descriptions are given for soil units that intersect or are mapped as occurring directly next to the Project alignment.
### TABLE 3 - PHYSICAL PROPERTIES OF TOPSOIL UNITS

<table>
<thead>
<tr>
<th>Unit #</th>
<th>Unit name</th>
<th>Major Components</th>
<th>Shrink-swell potential</th>
<th>Meets hydric criteria</th>
<th>Corrosion potential</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Steel</td>
</tr>
<tr>
<td>106</td>
<td>Baywood loamy sand, 15-30% slope</td>
<td>Baywood (85%)</td>
<td>low</td>
<td>no</td>
<td>low</td>
</tr>
<tr>
<td>124</td>
<td>Danville loam, 0 to 2% slope</td>
<td>Danville (85%)</td>
<td>moderate</td>
<td>no</td>
<td>moderate</td>
</tr>
<tr>
<td>130*</td>
<td>Elder sandy loam, 2 to 9% slope</td>
<td>Elder (85%)</td>
<td>low</td>
<td>no</td>
<td>low</td>
</tr>
<tr>
<td>133</td>
<td>Elkhorn sandy loam, 2 to 9% slope</td>
<td>Elkhorn (85%)</td>
<td>moderate</td>
<td>no</td>
<td>moderate</td>
</tr>
<tr>
<td>135</td>
<td>Elkhorn sandy loam, 15 to 30% slope</td>
<td>Elkhorn (85%)</td>
<td>moderate</td>
<td>no</td>
<td>moderate</td>
</tr>
<tr>
<td>136</td>
<td>Elkhorn-Pfeiffer complex, 30 to 50% slope</td>
<td>Elkhorn (45%)</td>
<td>moderate</td>
<td>no</td>
<td>moderate</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Concrete</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>143</td>
<td>Lompico-Felton complex, 30-50 % slopes</td>
<td>Lompico (40%)</td>
<td>low</td>
<td>no</td>
<td>low</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Felton (40%)</td>
</tr>
<tr>
<td>144*</td>
<td>Lompico-Felton complex, 50-75 % slopes</td>
<td>Lompico (45%)</td>
<td>low</td>
<td>no</td>
<td>low</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Felton (40%)</td>
</tr>
<tr>
<td>174</td>
<td>Tierra-Watsonville complex, 15-30% slope</td>
<td>Tierra (55%)</td>
<td>high</td>
<td>no</td>
<td>high</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Watsonville (30%)</td>
</tr>
<tr>
<td>175</td>
<td>Tierra-Watsonville complex, 30-50% slope</td>
<td>Tierra (50%)</td>
<td>high</td>
<td>no</td>
<td>high</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Watsonville (30%)</td>
</tr>
<tr>
<td>177</td>
<td>Watsonville loam, 2 to 15% slope</td>
<td>Watsonville (85%)</td>
<td>moderate</td>
<td>yes</td>
<td>moderate</td>
</tr>
</tbody>
</table>

*Unit is mapped as occurring directly next to and does not intersect the Project alignment.

Table 4 lists erosion factors for each soil unit found within 1,000 feet of the Project alignment. Erosion factor K indicates the susceptibility of a soil to sheet and rill erosion by water. Values of K range from 0.02 to 0.69; the higher the value, the more susceptible the soil is to sheet and rill erosion by water.

The erosion rating indicates the hazard of soil loss from unsurfaced roads and trails, and is based on soil erosion factor K, slope, and content of rock fragments. A rating of "slight" indicates that little or no erosion is likely; "moderate" indicates that some erosion is likely,
that the roads or trails may require occasional maintenance, and that simple erosion-control measures are needed; and "severe" indicates that significant erosion is expected, that the roads or trails require frequent maintenance, and that costly erosion-control measures are needed.

A wind erodibility group (WEG) consists of soils that have similar properties affecting their susceptibility to wind erosion in cultivated areas. The soils assigned to group 1 are the most susceptible to wind erosion, and those assigned to group 8 are the least susceptible.
### TABLE 4: EROSION PROPERTIES OF TOPSOIL UNITS

<table>
<thead>
<tr>
<th>Unit #</th>
<th>Unit name</th>
<th>Major Components</th>
<th>K factor</th>
<th>Erosion rating</th>
<th>Wind erodibility group</th>
</tr>
</thead>
<tbody>
<tr>
<td>106</td>
<td>Baywood loamy sand, 15-30% slope</td>
<td>Baywood (85%)</td>
<td>0.17</td>
<td>Severe</td>
<td>2</td>
</tr>
<tr>
<td>124</td>
<td>Danville loam, 0 to 2% slope</td>
<td>Danville (85%)</td>
<td>0.32</td>
<td>slight</td>
<td>6</td>
</tr>
<tr>
<td>130*</td>
<td>Elder sandy loam, 2 to 9% slope</td>
<td>Elder (85%)</td>
<td>0.17</td>
<td>moderate</td>
<td>3</td>
</tr>
<tr>
<td>133</td>
<td>Elkhorn sandy loam, 2 to 9% slope</td>
<td>Elkhorn (85%)</td>
<td>0.10</td>
<td>moderate</td>
<td>3</td>
</tr>
<tr>
<td>135</td>
<td>Elkhorn sandy loam, 15 to 30% slope</td>
<td>Elkhorn (85%)</td>
<td>0.10</td>
<td>severe</td>
<td>3</td>
</tr>
<tr>
<td>136</td>
<td>Elkhorn-Pfeiffer complex, 30 to 50% slope</td>
<td>Elkhorn (45%)</td>
<td>0.10</td>
<td>severe</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Pfeiffer (25%)</td>
<td>0.05</td>
<td>severe</td>
<td>5</td>
</tr>
<tr>
<td>143</td>
<td>Lompico-Felton complex, 30-50 % slopes</td>
<td>Lompico (40%)</td>
<td>0.37</td>
<td>severe</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Felton (40%)</td>
<td>0.17</td>
<td>severe</td>
<td>3</td>
</tr>
<tr>
<td>144*</td>
<td>Lompico-Felton complex, 50-75 % slopes</td>
<td>Lompico (45%)</td>
<td>0.37</td>
<td>severe</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Felton (40%)</td>
<td>0.17</td>
<td>severe</td>
<td>3</td>
</tr>
<tr>
<td>174</td>
<td>Tierra-Watsonville complex, 15-30% slope</td>
<td>Tierra (55%)</td>
<td>0.32</td>
<td>severe</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Watsonville (30%)</td>
<td>0.43</td>
<td>severe</td>
<td>5</td>
</tr>
<tr>
<td>175</td>
<td>Tierra-Watsonville complex, 30-50% slope</td>
<td>Tierra (50%)</td>
<td>0.32</td>
<td>severe</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Watsonville (30%)</td>
<td>0.43</td>
<td>severe</td>
<td>5</td>
</tr>
<tr>
<td>177</td>
<td>Watsonville loam, 2 to 15% slope</td>
<td>Watsonville (85%)</td>
<td>0.43</td>
<td>severe</td>
<td>5</td>
</tr>
</tbody>
</table>

*Unit is mapped as occurring directly next to and does not intersect the Project alignment.

Table 5 lists the typical topsoil profile of units that intersect or are located directly next to the Project alignment.
<table>
<thead>
<tr>
<th>Unit #</th>
<th>Unit name</th>
<th>Major Components</th>
<th>Depth (inches)</th>
<th>USC</th>
<th>USDA Texture</th>
</tr>
</thead>
<tbody>
<tr>
<td>106</td>
<td>Baywood loamy sand, Baywood (85%)</td>
<td>0 to 17</td>
<td>SM</td>
<td>Loamy sand</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>17-61</td>
<td>SM</td>
<td>Loamy fine sand, loamy sand</td>
<td></td>
</tr>
<tr>
<td>124</td>
<td>Danville loam, 0 to 2% slope Danville (85%)</td>
<td>0-17</td>
<td>CL, CL-ML, SC, SC-SM</td>
<td>Loam</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>17-29</td>
<td>CH, CL</td>
<td>Clay, sandy clay, silty clay</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>29-65</td>
<td>SC</td>
<td>Clay loam, gravelly sandy clay loam, sandy clay loam</td>
<td></td>
</tr>
<tr>
<td>130*</td>
<td>Elder sandy loam, 2 to 9% slope Elder (85%)</td>
<td>0-21</td>
<td>SC</td>
<td>Sandy loam</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>21-67</td>
<td>SC</td>
<td>Clay loam, gravelly sandy clay loam, sandy clay loam</td>
<td></td>
</tr>
<tr>
<td>133</td>
<td>Elkhorn sandy loam, 2 to 9% Elkhorn (85%)</td>
<td>0-21</td>
<td>SM</td>
<td>Sandy loam</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>21-61</td>
<td>CL, SC</td>
<td>Clay loam, sandy clay loam</td>
<td></td>
</tr>
<tr>
<td>135</td>
<td>Elkhorn sandy loam, 15 to 30% slope Elkhorn (85%)</td>
<td>0-21</td>
<td>SM</td>
<td>Sandy loam</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>21-61</td>
<td>CL, SC</td>
<td>Clay loam, sandy clay loam</td>
<td></td>
</tr>
<tr>
<td>136</td>
<td>Elkhorn-Pfeiffer complex, 30 to 50% slope Elkhorn (45%)</td>
<td>0-21</td>
<td>SM</td>
<td>Sandy loam</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>21-61</td>
<td>CL, SC</td>
<td>Clay loam, sandy clay loam</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>0-24</td>
<td>SM</td>
<td>Gravelly sandy loam</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>24-66</td>
<td>SM</td>
<td>Coarse sandy loam, gravelly coarse sandy loam, gravelly sandy loam</td>
<td></td>
</tr>
<tr>
<td>143</td>
<td>Lompico-Felton complex, 30-50 % slopes</td>
<td>0-14</td>
<td>CL</td>
<td>Loam</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>14-30</td>
<td>CL, SC</td>
<td>Clay loam, loam, sandy clay</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>30-37</td>
<td>CL, SC</td>
<td>Clay loam, extremely gravelly sandy clay loam, loam, sandy clay loam</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>0-11</td>
<td>SM</td>
<td>Sandy loam</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>11-43</td>
<td>CL, SC</td>
<td>Clay loam, sandy clay loam, silty clay loam</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>43-63</td>
<td>CL-ML, SC, SC-SM, SM</td>
<td>Loam, sandy clay loam, sandy loam</td>
<td></td>
</tr>
<tr>
<td>144*</td>
<td>Lompico-Felton complex, 50-75 % slopes</td>
<td>0-14</td>
<td>CL</td>
<td>Loam</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>14-30</td>
<td>CL, SC</td>
<td>Clay loam, loam, sandy clay</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>30-37</td>
<td>CL, SC</td>
<td>Clay loam, extremely gravelly sandy clay loam, loam, sandy clay loam</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>0-11</td>
<td>SM</td>
<td>Sandy loam</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>11-43</td>
<td>CL, SC</td>
<td>Clay loam, sandy clay loam, silty clay loam</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>43-63</td>
<td>CL-ML, SC, SC-SM, SM</td>
<td>Loam, sandy clay loam, sandy loam</td>
<td></td>
</tr>
<tr>
<td>Unit #</td>
<td>Unit name</td>
<td>Major Components</td>
<td>Depth (inches)</td>
<td>USC</td>
<td>USDA Texture</td>
</tr>
<tr>
<td>-------</td>
<td>-----------</td>
<td>------------------</td>
<td>---------------</td>
<td>-----</td>
<td>--------------</td>
</tr>
<tr>
<td>174</td>
<td>Tierra-Watsonville complex, 15-30% slope</td>
<td>Tierra (55%)</td>
<td>0-14</td>
<td>SC-SM, SM</td>
<td>Sandy loam</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>14-66</td>
<td>CH, CL</td>
<td>Clay, clay loam, sandy clay</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Watsonville (30%)</td>
<td>0-18</td>
<td>ML</td>
<td>Loam</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>18-39</td>
<td>CH, CL</td>
<td>Clay, clay loam</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>39-63</td>
<td>CL, SC</td>
<td>Clay loam, sandy clay loam</td>
</tr>
<tr>
<td>175</td>
<td>Tierra-Watsonville complex, 30-50% slope</td>
<td>Tierra (50%)</td>
<td>0-14</td>
<td>SC-SM, SM</td>
<td>Sandy loam</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>14-66</td>
<td>CH, CL</td>
<td>Clay, clay loam, sandy clay</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Watsonville (30%)</td>
<td>0-18</td>
<td>ML</td>
<td>Loam</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>18-39</td>
<td>CH, CL</td>
<td>Clay, clay loam</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>39-63</td>
<td>CL, SC</td>
<td>Clay loam, sandy clay loam</td>
</tr>
<tr>
<td>177</td>
<td>Watsonville loam, 2 to 15% slope</td>
<td>Watsonville (85%)</td>
<td>0-18</td>
<td>ML</td>
<td>Loam</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>18-39</td>
<td>CH, CL</td>
<td>Clay, clay loam</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>39-63</td>
<td>CL, SC</td>
<td>Clay loam, sandy clay loam</td>
</tr>
</tbody>
</table>

*Unit is mapped as occurring directly next to and does not intersect the Project alignment.

Onsite investigation may be needed to validate topsoil data presented in this section and to confirm the identity of the soil on a given site.

### 3.3 Surface Conditions

The project alignment appears to be built upon Pleistocene lowest emergent coastal terrace deposits and cut and/or filled where drainages or over/underpasses occur. Topography is relatively flat with creeks incising into Pleistocene lowest emergent coastal terrace deposits. The main creeks drain south from Pleistocene coastal terrace deposits and the Santa Cruz Mountains and have typically incised into the Pleistocene lowest emergent coastal terrace deposits about 40 feet to 55 feet. Elevation increases along the alignment from west to east with a minimum and maximum of about 33 feet and 166 feet respectively. These elevations are based on NAVD 88 vertical datum.

Prior to urbanization, and construction of SR-1, land use of the project area appeared to be agricultural. Current land use adjacent to the project is mostly single level residential with occasional areas of open grass land and woodlands.
### 3.4 Subsurface Conditions

Based on the As-Built LOTBs of the existing structures within the project alignment, following is the subsurface soil conditions at each location:

<table>
<thead>
<tr>
<th>Location</th>
<th>Referred Boring Number</th>
<th>As-built Boring Ground Surface Elevation (feet)</th>
<th>Generalized Soil Conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Retaining Wall at Proposed Interchange at Freedom Blvd (As-Built LOTB, dated 8/15/1967)</td>
<td>B-3</td>
<td>225.1</td>
<td>Very dense silty fine to coarse Sand up to the maximum explored depth of 70 feet (Elev. 155.1 feet).</td>
</tr>
<tr>
<td>Rio Del Mar Boulevard OC (Bride No. 36-23) (As-Built LOTB, dated 7/9/1963)</td>
<td>B-2</td>
<td>130.2</td>
<td>Slightly compact to compact silty fine to medium sand in upper 4 feet (Elev. 126.2 ft), underlain by dense to very dense silty fine to coarse sand up to the maximum explored depth of 27.5 feet (Elev. 102.7 ft)</td>
</tr>
<tr>
<td>State Park Dr Overcrossing (Bride No. 36-28) (As-Built LOTB, dated April, 1961)</td>
<td>B-1, B-2</td>
<td>132.9, 142.3</td>
<td>Loose to very dense fine to medium sand, well graded sand, gravel, clayey sand and some clay up to the maximum explored depth of 40.5 feet (Elev.101.8 ft).</td>
</tr>
<tr>
<td>Rob Roy Junction OC (Bride No. 36-0022) (As-Built LOTB 1963)</td>
<td>B-2</td>
<td>146.2</td>
<td>Slightly compact fine to coarse sand with layers of granule gravel in upper 7 feet (Elev. 139.2ft.) underlain by very loose, very fine sand up to depth of 12.5 ft (Elev. 133.7 ft.) underlain by dense brown medium sand up to the maximum explored depth of 40 feet (Elev. 106 feet).</td>
</tr>
</tbody>
</table>

The locations of as-built structures are shown in Plane No.5, As-Built Boring Location Map. The subsurface soil conditions along the project alignment should be verified during the PS&E phase.

### 3.5 Groundwater

Groundwater levels shown on the As-Built and Reference LOTBs are summarized in the table below.
### TABLE 7: SUMMARY OF GROUNDWATER LEVELS

<table>
<thead>
<tr>
<th>As-Built LOTB Borehole No. (Year)</th>
<th>Ground Surface Elevation, (feet)</th>
<th>Boring/ Penetration Depth (feet)</th>
<th>Groundwater Table or Piezometric Elevation</th>
<th>Date Measured</th>
<th>As-Built LOTB Project Name</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1 (1963)</td>
<td>130.1</td>
<td>11.5</td>
<td>Not Encountered</td>
<td>NA</td>
<td>Rio Del Mar Boulevard OC (Bride No. 36-23)</td>
</tr>
<tr>
<td>B-2 (1963)</td>
<td>130.2</td>
<td>26</td>
<td></td>
<td>06/04/1963</td>
<td>Retaining Wall at Proposed Interchange at Freedom Blvd</td>
</tr>
<tr>
<td>B-3 (1963)</td>
<td>132.9</td>
<td>8.8</td>
<td></td>
<td>08/08/1967</td>
<td></td>
</tr>
<tr>
<td>B-1 (1967)</td>
<td>219.3</td>
<td>42</td>
<td></td>
<td>06/04/1967</td>
<td>Retaining Wall at Proposed Interchange at Freedom Blvd</td>
</tr>
<tr>
<td>B-2 (1967)</td>
<td>222.1</td>
<td>52.1</td>
<td></td>
<td>08/08/1967</td>
<td></td>
</tr>
<tr>
<td>B-3 (1967)</td>
<td>225.1</td>
<td>70</td>
<td></td>
<td>08/08/1967</td>
<td></td>
</tr>
<tr>
<td>B-4 (1967)</td>
<td>225.4</td>
<td>52.1</td>
<td></td>
<td>08/08/1967</td>
<td></td>
</tr>
<tr>
<td>B-1 (1963)</td>
<td>151.8</td>
<td>58</td>
<td>20.0</td>
<td>131.8</td>
<td></td>
</tr>
<tr>
<td>B-2 (1963)</td>
<td>146.2</td>
<td>40.4</td>
<td>Not Encountered</td>
<td>06/04/1963</td>
<td>Rob Roy Junction OC (Bride No. 36-22)</td>
</tr>
<tr>
<td>B-3 (1963)</td>
<td>140.0</td>
<td>40</td>
<td>3.5</td>
<td>136.5</td>
<td></td>
</tr>
<tr>
<td>B-1 (1961)</td>
<td>132.9</td>
<td>26.9</td>
<td></td>
<td>04/25/61</td>
<td>State Park Dr Overcrossing (Bride No. 36-28)</td>
</tr>
<tr>
<td>B-2 (1961)</td>
<td>142.3</td>
<td>40.5</td>
<td></td>
<td>04/25/61</td>
<td></td>
</tr>
<tr>
<td>B-3 (1961)</td>
<td>142.6</td>
<td>22.5</td>
<td></td>
<td>04/25/61</td>
<td></td>
</tr>
<tr>
<td>B-4 (1961)</td>
<td>141.3</td>
<td>20.7</td>
<td></td>
<td>04/27/61</td>
<td></td>
</tr>
<tr>
<td>B-5 (1961)</td>
<td>141.7</td>
<td>31.2</td>
<td></td>
<td>04/25/61</td>
<td></td>
</tr>
</tbody>
</table>

The groundwater level is anticipated to vary with the passage of time due to seasonal groundwater fluctuation, surface and subsurface flows into nearby water course, ground surface run-off, and other environmental factors that may not be present at the time of the previous field exploration. As discussed in Section 3.3, highest and lowest ground surface elevation varies between 33 feet and 166 feet along the project alignment, which is about 2.6 mile long. Therefore, it is not recommended to consider same groundwater elevation for the entire project alignment based on limited subsurface data. Based on our research, historical groundwater data are not available along the project alignment. Based on our field exploration (2020 & 2021) for the State Park to Bay Ave project (located west of current project limit), maximum ground water was encountered below 15 feet from existing surface with similar ground surface elevation of the current project.
Therefore, it is recommended preliminary design groundwater depth of 15 feet (elevations varies along project alignment) from the surface for preliminary analysis. The groundwater conditions at the project site should be verified during the PS&E phase.

3.6 Seismic Hazards

3.6.1 Ground Motion Parameters

Regional Seismicity
The regional seismic context is an important consideration because the forces that affect the project area are regional in nature: that is, they are generated off-site, outside the immediate area, or outside Santa Cruz County. However, the effects of these forces must be accommodated within the limits of the project, in compliance with regulations and guidelines established by the State and County.

The project site is located in Santa Cruz County and lies within one of the most seismically active areas of the United States. The area is influenced mostly by the San Andreas fault system, which spans the Coast Ranges from the Pacific Ocean to the San Joaquin Valley. The project location is between two major active faults, the San Andreas and San Gregorio, approximately 6.1 miles north-east and 15.2 miles south-west to the project site, respectively. A fault map of the area is shown on Plate 2.

Acceleration Response Spectra
The recommended design response spectrum for the proposed overcrossing structure was determined using the Caltrans ARS Online tool (V3.0.2) which is consistent with the Caltrans SDC V2.0.

For SDC 2.0, the Design Spectrum is based on the USGS 975-year uniform hazard spectrum only. Effective December 1, 2019, the USGS hazard spectrum is based on the 2014 National Hazard Map per the memorandum from the State Bridge Engineer. The updated Design Spectrum continues the use of near-fault adjustment factors and basin amplification factors. The only change to these factors is the use of the Campbell-
Bozorgnia (2014) and Chiou-Youngs (2014) basin amplification factors, updated from their 2008 models.

The development of the design ARS curve is based on several input parameters, including site location (longitude/latitude), average shear wave velocity for the top 30m/100 feet (Vs30m), and other site parameters, such as fault characteristics and site-to-fault distances.

Average shear wave velocities for the top 100 feet of soils at the site were estimated by using established correlations and procedure provided in Caltrans Geotechnical Manual “Design Acceleration Response Spectrum” module, January 2021. Shear wave velocity calculations are attached in Appendix V.

Based on the subsurface data, the site is classified as “Class S2 Soil” per Caltrans SDC, V2.0. The site location and the relevant parameters are summarized in the table below, and the recommended design curve is presented in Plate 6:

| TABLE 8 – RECOMMENDED GROUND MOTION PARAMETER FOR GEOTECHNICAL DESIGN |
|-------------------------------|---------------------------------|-----------------|----------------|----------------|-----------------|
|                              | Site Parameter                  | Locations       | Shear-Wave Velocity $V_{s30}$, m/sec | Horizontal Peak Ground Acceleration (HPGA) ($g$) | Mean Earthquake Moment Magnitude | Mean Site-to-Fault/ Rupture Surface Distance Rrup, km |
| Latitudes degrees            | Longitude, degrees              | 36.9754         | -121.8905 | 239            | 0.71            | 6.97             | 13.2            |

1. Based on the Caltrans web tool ARS Online (Version 3.0.2)
2. An adjustment factor for the near-fault effect was applied to the calculated spectral acceleration values. The increase of 20% to the spectral acceleration values corresponds to period longer than 1 second and linearly tapers to zero at a period of 0.5 seconds.
3. No adjustment was needed for basin effect.

### 3.6.2 Parameters for Seismic Slope Stability Analysis

Design Horizontal Seismic Coefficient of 0.24g is recommended for the pseudo-static slope stability analysis.
3.6.3 Fault Rupture

The majority of the alignment lays within the Soquel 7.5 minute Quadrangle which is not in the Alquist-Priolo Earthquake Fault Zone. The far eastern end of the proposed project, approximately 1,200 feet, lays within the Watsonville West 7.5 minute Quadrangle and shows that this project section does not lay within an Alquist-Priolo Earthquake Fault Zone.

The USGS Quaternary Fault and Fold Database does not show any faults aged less than 15,000 years within 1,000 feet of the Project. Plate 2 shows the location of faults in relation to the project. The Zayante-Vergules Lower fault appears to strike through the eastern section of the Project. The fault has been included in Plate 2 because the fault is listed in the Caltrans Fault Database V2. A literature review conducted as part of this report has not found sufficient information to further characterize the location and age of the Zayante-Vergules Lower fault.

Considering the above paragraphs, the potential for ground surface rupture due to faulting does not exist. This statement does not preclude the existence of unknown active faults within the Project limits.

3.6.4 Liquefaction

Liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary but essentially total loss of shear strength under the reversing, cyclic shear stresses associated with earthquake shaking. Submerged cohesionless sands and silts of low relative density are the type of soils, which usually are susceptible to liquefaction. Clays are generally not susceptible to liquefaction.

The liquefaction potential was evaluated in accordance with the methods proposed in Liquefaction Evaluation Module of Caltrans Geotechnical Manual (January 2020). The evaluation was done using the boring data from all the available borings using a Magnitude 7.07 earthquake and a peak ground acceleration of about 0.74g. This method compares the estimates of the earthquake-induced shear stress to the susceptibility of soil liquefaction. According to Bray (2006), liquefaction appears to occur in soils where
these fines are either non-plastic or are low plasticity silts and/or silty clays (PI<12%, and LL<37%), and with high water content relative to their liquid limit (w>0.85 LL).

The soil data from as-built and Reference LOTBs were analyzed per Youd (2001) and the analysis results are presented in Appendix V.

Based on our preliminary liquefaction analysis of as-built subsurface data, one potential liquefiable layer (between depth 13 and 18 feet from the surface) was identified in Roy Junction OC As-Built boring log. Therefore, based on the analysis of limited subsurface data, the liquefaction potential exists along the project alignment. The liquefaction potential needs to be studied further in the PS&E phase based on additional subsurface data. If liquefaction potential exists, the loss of strength due to liquefaction should be considered in the design.

3.6.5 Liquefaction-Induced Lateral Spreading

Per Caltrans Memo to Designers 20-15 dated May 2017, “Lateral Spreading Analysis for New and Existing Bridges”, lateral spreading is caused by the accumulation of incremental displacements that develop within liquefied soil under cyclic loading. Depending on the number and amplitude of stress pulses, lateral spreading can produce displacements that range from a few inches to tens of feet. Flow liquefaction occurs when a slope becomes unstable under static loading due to strength loss caused by liquefaction.

Lateral spreading refers to the more moderate movements of gently sloping ground due to soil liquefaction. The geologic conditions conducive to lateral spreading (gentle surface slope, shallow water table, and liquefiable cohesionless soils) are frequently found along streams and other waterfronts in recent alluvial or deltaic deposits, as well as in loosely-packed, saturated, sandy fills.

In our opinion, the potential for lateral spreading does not exist because continues layers of potential liquefiable layers were not identified at the project site. The lateral spreading potential needs to be studied further in the PS&E phase based on additional subsurface data.
3.6.6 Tsunami Inundation

Tsunamis are large ocean waves generated by major seismic events. According to the Soquel Tsunami Inundation Map for Emergency Planning, Aptos Creek is susceptible to tsunami inundation. The area of inundation susceptibility is shown in Plate 4.

Based on Memo to Designers 20-13 (MTD 20-13), the tsunami hazard is significantly reduced at locations beyond one-half mile of the coast or at elevation greater than 40 feet above mean sea level. The project alignment is between 0.45 and 1.25 mile from the coast. As discussed in section 3.3, elevation increases along the alignment from west to east with a minimum and maximum of about 33 feet and 166 feet, respectively. Since portion of the project is less than one-half mile and below 30 feet elevation, tsunami hazard effect on structures should be considered during PS&E phase of the project.

4.0 GEOTECHNICAL DESIGN EVALUATION

4.1 Existing Slopes - Landslides

Landslides occur when shear stress in a soil or rock mass exceeds shear strength. Shear stress can be increased by adding to the weight of soil or rock mass through saturation or loading. Shear strength can be reduced by erosion or by grading at the toe of a slide mass. Slope failure can be caused by an increase in shear stress or a decrease in shear strength. Zones of low shear strength are often associated with the presence of expansive clays and weak bedrock units. Earthquake-induced ground-shaking can cause activation of new or previously existing landslides and other slope instabilities, especially during periods of high groundwater.

As shown on Plate No. II-1, the geomorphology of the alignment is dominated by coastal terraces with creeks incising down to alluvial covered bedrock; the steepest natural slopes are found along creek banks. The steepest slopes of the alignment appear to be engineered.

Geological mapping does not indicate the presence of historical or quaternary landslides along the project alignment. However, several Quaternary landslides are mapped within
about 300 to 500 feet from the southern part of the alignment and are shown on the Plate II-1. These landslides do not appear to be a potential hazard to the project.

4.2 Earthwork and Grading

Based on the preliminary plans we understand that project requires cut and fill for the roadway widenings and retaining wall construction. Areas to receive engineered fill or structure backfill should be excavated to remove any loose/soft soil materials. The resulting surface upon which fill is to be placed should be observed by the Geotechnical Engineer. Areas receiving fill should be scarified, moisture conditioned and compacted in accordance with Caltrans standard specifications. In general, engineered fill or structure backfill imported to the site should be relatively non-expansive granular material having a Plasticity Index (P.I.) less than 15, a minimum Sand Equivalent of 10, clean and free of debris and organic material, and should be reviewed by the Geotechnical Engineer.

Engineered fill should have a minimum 90-percent relative compaction per Caltrans standard (Section 19, Caltrans Standard Specifications, 2018) except that 95-percent compaction is recommended for the upper 6-inch of the pavement subgrade and foundation subgrade of the structures. The extent of the 95-percent compaction for the pavement subgrade should be followed as specified in Caltrans 2018 Standard Specifications, Section 19-5.03B:

a) 0.5 feet below the grading plane for the width between the outer edges of shoulder.

b) 2.5 feet below the finished grade for the width of the traveled way plus 3 feet on each side.

The structure foundation subgrade excavation and fill compaction requirement should be in accordance with Caltrans 2018 Standard Specifications, Section 19-3 “Structure Excavation and Backfill”.

In case high groundwater level and soft/weak subsoils are encountered, the use of geotextile, geogrid and additional aggregate base rock should be anticipated to build a working platform to facilitate subgrade preparation. Additional engineering analyses and design options may also be required to help mitigate foundation support failure, if needed.

The on-site materials exposed after the excavation may be used for engineered fill
provided that they meet the above criteria (P.I. and Sand Equivalent) and are not contaminated.

4.3 Expansive Soil

As discussed in section 3.4, expansive clays were not encountered near surface in the as-built borings. It should be verified during PS&E phase. If expansive soils are encountered during PS&E phase field investigation, it is recommended to perform laboratory tests such as Plasticity Index, Expansion Index, and R-value to investigate the expansive soil properties of the subsurface soils underlying the project site. There will be an impact on the structural pavement design and/or shallow footings if expansive soil is encountered in the pavement subgrade or footing subgrade.

4.4 Construction Consideration

Construction Activities

Construction work for the Build Alternative would be done primarily during daylight hours from 7:00 a.m. to 6:00 p.m. However, night-time work and temporary closures of lanes and roadways may be necessary to avoid major disruption for tasks that could interfere with traffic or create safety hazards such as demolition of the existing railroad bridges, construction and removal of falsework, and lifting and placing new railroad bridges and pedestrian overcrossings. Construction activities would include excavation, drilling, dewatering, pavement demolition, bridge demolition, mass grading, concrete form work, pavement installation, storm system installation, landscaping and irrigation, sign installation, striping operations, and traffic control. Such activities would require the use of the following types of equipment: drilling rig, forklift, scissor lift, backhoe, track excavator, compactor, concrete pump, crane, bulldozer, grader, front-end loader, dump trucks, jackhammer, and vibratory roller. These activities may require temporary freeway, ramp, and local street partial lane closures or full closures with possible detours.

A Transportation Management Plan (TMP) would be developed as part of the project construction planning phase. The TMP would address potential impacts to circulation of all modes of travel (i.e., transit, bicycles, pedestrians, and vehicles). Roadway and/or pedestrian access to all occupied businesses and respective parking lots would be
maintained during project construction. The TMP would include an evaluation of potential detour impacts and would also include measures to minimize, avoid, and/or mitigate impacts to alternate routes. The TMP would address coordination with local agencies for traffic through or near the construction zone. Staging areas would be located within the existing Caltrans right-of-way and within the Santa Cruz branch line right-of-way along Coastal Rail Trail Segment 12, where feasible.

**Construction Schedule**
Construction of the SR 1 improvements including the auxiliary lanes and BOS features is anticipated to begin in 2025 and is estimated to take approximately 3 years to complete. The construction schedule for the Coastal Rail Trail Segment 12 is not determined at this time. Optional Phase 1 Interim Improvements could be constructed initially with the SR 1 improvements and could remain in place until such time that funding is available for Phase 2 ultimate improvements. The timing for Phase 2 ultimate improvements will be determined as future funding becomes available.

**Demolition**
Demolition work would generally comprise removal of existing bridge structures, abutments, columns, overhead sign foundations, rails and ties, clearing and grubbing, tree removal, pavement removal, and drainage system removal.

**Stormwater Drainage and Treatment Facilities**
The Build Alternative would include drainage system improvements and permanent stormwater treatment facilities for the SR 1 and Coastal Rail Trail Segment 12 improvements. Hydromodification measures would be included, if needed. During construction, the contractor would be required to develop and implement a Storm Water Pollution Prevention Plan (SWPPP) in compliance with the statewide Construction General Permit and consistent with the guidelines and procedures in Caltrans’ Statewide Storm Water Management Plan. The SWPPP will provide detailed, site-specific information regarding best management practices to avoid and minimize water quality impacts. The project would be constructed to minimize erosion by disturbing slopes only when necessary, minimizing cut and fill areas to reduce slope lengths, providing cut and
fill slopes flat enough to allow revegetation to limit erosion rates, and providing concentrated flow conveyance systems such as storm drains, ditches, and gutters.

**Utilities**
Existing utilities located in areas subject to construction that conflict with the proposed improvements would be relocated as needed. This is anticipated to include sanitary sewer and electric utility poles adjacent to Moosehead Drive and a gas line along the Coastal Rail Trail Segment 12 route for the Phase 2 ultimate improvements.

**Property Acquisitions**
The Build Alternative would require full or partial acquisitions for the construction of the SR 1 and Coastal Rail Trail Segment 12 improvements, as well as temporary easements for construction activities such as the construction of sound walls and retaining walls along SR 1.

5.0 **RECOMMENDATIONS**

5.1 **Recommendations for Future Exploration and Investigations (Sound Walls and Overhead Signs)**

We prepared separate Structural Preliminary Foundation Reports (SPGR) for the bridge structures and nonstandard retaining walls. Future exploration recommendations for those structures are discussed in the relevant SPGR. Future explorations for the sound wall and overhead sign structures are discussed in this section.

Borings are proposed to provide information regarding subsurface soil conditions and groundwater conditions for the Project. During the PS&E phase, geotechnical investigations should be conducted to evaluate the engineering properties of the subsurface soil materials for recommendation of geotechnical parameters and to address geotechnical hazards associated with different design elements (such as slope stability and settlement etc.) and hazards associated with strong ground motion (shaking and liquefaction, etc.).
In general, deep borings with undisturbed samples would be required for high fill, deep cut and retaining structures for settlement, stability, and support evaluation. Shallow borings are needed for roadway/pavement design. The lab testing would be done taking into account both foundation and roadway improvements for borings that would be shared.

Auger borings and/or rotary-wash borings are proposed. Shelby samples (push) would be taken if very soft to soft clay layers are encountered during field investigations. Field decisions may have to be made based on what site conditions are encountered.

The approximate locations of the borings are shown in Plate 7, Proposed Boring Location Map. Caltrans As-Built borings would be referred to near the existing structures. Below is a summary table for the proposed future field exploration for sound walls:

<table>
<thead>
<tr>
<th>Sound Wall No.</th>
<th>Approximate Length (ft)</th>
<th>Boring ID</th>
<th>Boring Depth (feet)</th>
<th>Site Access/Traffic Control Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>S68</td>
<td>3,293</td>
<td>S68-1</td>
<td>40</td>
<td>Eastbound Soquel Drive shoulder/traffic control of shifting lanes with cones. One additional boring (OH-4) will be shared overhead sign boring.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>S68-2</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>S68-3</td>
<td></td>
<td></td>
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<tr>
<td></td>
<td></td>
<td>S68-4</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>S68-5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>S71</td>
<td>3,280</td>
<td>S71-1</td>
<td>40</td>
<td>Southbound SR 1 right shoulder/traffic control of shoulder closure and/or right lane closure.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>S71-2</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>S71-3</td>
<td></td>
<td></td>
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<tr>
<td></td>
<td></td>
<td>S71-4</td>
<td></td>
<td></td>
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<tr>
<td></td>
<td></td>
<td>S71-5</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>S71-6</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>S71-7</td>
<td>40</td>
<td>Next to the fence on the top of slope at Rio Del Mar Blvd on-ramp.</td>
</tr>
<tr>
<td>SB-1</td>
<td>141</td>
<td>SB1-1</td>
<td>40</td>
<td>Next to the fence on the top of the slope</td>
</tr>
<tr>
<td>S86a</td>
<td>606</td>
<td>S86a-1</td>
<td>40</td>
<td>Next to the fence on the top of the slope, one additional boring (R-22-003) will be shared with south Aptos POC boring.</td>
</tr>
<tr>
<td>Sound Wall No.</td>
<td>Approximate Length (ft)</td>
<td>Boring ID</td>
<td>Boring Depth (feet)</td>
<td>Site Access/Traffic Control Notes</td>
</tr>
<tr>
<td>----------------</td>
<td>-------------------------</td>
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<td>--------------------</td>
<td>----------------------------------</td>
</tr>
<tr>
<td>S87</td>
<td>1,057</td>
<td>S87-1</td>
<td>40</td>
<td>Next to the fence on the top of the slope. Two additional borings (R-22-013 &amp; R-21-017) will be shared with retaining wall.</td>
</tr>
<tr>
<td>S89</td>
<td>885</td>
<td>S89-1</td>
<td>40</td>
<td>Southbound SR 1 right shoulder/traffic control of shoulder closure and/or right lane closure. Three additional borings (R-22-005, R-22-007 &amp; R-21-019) will be shared with Aptos Creek bridge widening and retaining wall.</td>
</tr>
<tr>
<td>S93</td>
<td>585</td>
<td>S93-1</td>
<td>40</td>
<td>Next to the fence on the top of the slope. One additional boring (R-22-001) will be shared with North Aptos POC boring.</td>
</tr>
<tr>
<td>S90</td>
<td>1,862</td>
<td>S90-1</td>
<td>40</td>
<td>Northbound SR 1 right shoulder/traffic control of shoulder closure and/or right lane closure.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>S90-2</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>S90-3</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>S90-4</td>
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</tbody>
</table>

The soil samples for these borings would be tested in the laboratory to determine their water content, unit weight, strength, gradation, consolidation, corrosion, and plasticity properties, as needed. Project specific laboratory test program would be developed after the samples are reviewed by the engineer in the laboratory.

One boring to the depth between 40-50 feet will be drilled at each overhead sign location. Boring will be drilled at the proposed OH sign location or within 50 feet if access is an issue. No instrumentation is proposed.

5.2 Fill Slopes/Embankments

**General**
Areas to receive engineered fill should be excavated to remove any loose/soft soil materials. The resulting surface upon which fill is to be placed should be observed by the Geotechnical Engineer. Areas receiving fill should be scarified, moisture conditioned and compacted in accordance with Caltrans standard specifications.

**Fill Material**
In general, engineered fill imported to the site should be relatively non-expansive granular material having a Plasticity Index (P.I.) less than 15, a minimum Sand Equivalent of 10,
clean and free of debris and organic material and should be reviewed by the Geotechnical Engineer. Engineered fill should have a minimum 90-percent relative compaction per Caltrans standard except that 95-percent compaction is recommended for the upper 6-inch of the pavement subgrade and foundation subgrade of the structures. The on-site materials exposed after the excavation may be used for engineered fill provided that they meet the above criteria (P.I. and Sand Equivalent) and are not contaminated as discussed in Section 4.2 of the report.

**Settlement**

Settlement in the cohesionless material is expected to occur relatively fast. However, fine-grained materials such as clays may require longer periods. Site-specific ground investigation in the PS&E phase will be required to confirm the subsurface soil conditions for the future embankments. If further investigation shows that consolidation settlement become critical, mitigation measures such as staged construction, implementing waiting periods, surcharge fill and ground improvement technique such as installation of wick drains prior to the fill placement would be required and settlement monitoring will be required during the construction period. Some of these mitigation measures might impact the project schedule and should be addressed early in the PS&E phase.

**Stability**

In our opinion, the fill slopes should not be steeper than 2(H): 1(V). Should there be no constraints from existing conditions, a flatter slope gradient such as the 4(H):1(V) may be required according to the “Highway Design Manual Topic 304 – Side Slopes”. Proper drainage and erosion control measures are therefore important to maintain the overall stability of the slopes. Regular slope maintenance is important and should be incorporated in the project plans. Landscaping should be planned to protect the new slopes. A detailed study should be conducted to analyze the slope stability of specific slopes that are developed for the project.

**Fill Placement**

Foundation of embankments should be prepared in accordance with Caltrans Standard for “Clearing and Grubbing” and “Earthwork”. The embankment fill should be placed in accordance with the guidelines provided in the Caltrans Highway Design Manual. These
guidelines require structure approach embankment material to be compacted to a “95% Relative Compaction”. This also reduces the potential for earthquake-induced settlement or slippage to occur.

Fills to be placed on existing slope should be keyed and benched into the slope. The maximum height of the key should be typically between 3 feet and 4 feet. The minimum width of the key should be 6 feet. For the fill to be placed on the existing slope (not behind the retaining wall), it is recommended that the fill to be placed on the slopes should be over-built and cut back to the proposed grade. Appropriate subdrain systems should be provided within the fill slopes to mitigate subsurface seepage.

5.3 Cuts
Significant cut is considered at along SR 1 for the highway widening and retaining wall construction. 2H:1V slope is recommended for the roadway widening cut slopes and 1.5H:1V temporary slope is recommended for retaining wall construction. Slope stability analysis should be performed during PS&E phase.

5.4 Retaining Walls
As discussed in Section 1.1.3, we understand that nine nonstandard retaining walls are proposed for the project. We understand that Sub Horizontal Ground Anchors (SHGA), soil nail, MSE and Caltrans Standard Type 7B walls are considered for the project. Based on anticipated subsurface conditions, considered type of walls are feasible on geotechnical standpoint. Geotechnical design parameters for the wall design will be provided during PS&E phase based on site specific subsurface investigation.

5.5 Sound Walls and Overhead Signs
Geotechnical design recommendations for sound wall and overhead signs will be provided during PS&E phase.

5.6 Summary of Recommendations
If the designer has questions or concerns with any of these recommendations, or, if conditions are found to be different during construction, the Geotechnical Engineer who
prepared this report should be contacted. A concise summary of the geotechnical recommendations is presented below:

- Horizontal peak ground acceleration (HPGA) at the project location is 0.71g (Ref.: Section 3.6.1).
- Liquefaction potential exists along project alignment based on existing subsurface data. It should be verified during PS&E Phase (Ref.: Section 3.6.3).
- Based on the available boring information and soil survey map, potential expansive soils may not exist at the site. Laboratory tests such as Plasticity Index, and expansion index should be performed in the PS&E phase to confirm the expansive soil properties of the subsurface soils underlying the project site (Ref.: Section 4.3).
- Caltrans Standard, MSE and SHGA walls can be considered for retaining walls. Settlement should be evaluated based on subsurface data at the fill location during PS&E phase (Ref.: Section 5.4).
6.0 REFERENCES

The documents and websites referred to in this report are summarized below:

1) Caltrans Department of Transportation (Caltrans), November 2012, “Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendations”.


7.0 REPORT COPY LIST

This report is prepared for Mark Thomas (Designer) as a part of Highway 1 Aux Lanes Project in the County of Santa Cruz, California. A copy of this report will be submitted to the designer. The report should be distributed to those listed under Report Distribution in the Communications and Reporting section of Offices of Geotechnical Design – Quality Management Plan. A geotechnical report distribution list is provided below as reference.

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</tbody>
</table>
8.0 INVESTIGATION LIMITATIONS

Our services consist of professional opinions and recommendations made in accordance with generally accepted geotechnical engineering principles and practices and are based on available subsurface data and the assumption that the subsurface conditions do not deviate from available data. All work done is in accordance with generally accepted geotechnical engineering principles and practices. No warranty, expressed or implied, of merchantability or fitness, is made or intended in connection with our work or by the furnishing of oral or written reports or findings.

The scope of our services did not include any environmental assessment or investigation for the presence or absence of hazardous or toxic materials in structures, soil, surface water, groundwater, or air, below or around this site. Unanticipated soil conditions are commonly encountered and cannot be fully determined by taking soil samples and excavating test borings; different soil conditions may require that additional expenditures be made during construction to attain a properly constructed project. Some contingency fund is thus recommended to accommodate these possible extra costs.

This report has been prepared for the proposed project as described earlier, to assist the engineer in the design of this project. In the event any changes in the design or location of the facilities are planned, or if any variations or undesirable conditions are encountered during construction, our conclusions and recommendations shall not be considered valid unless the changes or variations are reviewed, and our recommendations modified or approved by us in writing.

This report is issued with the understanding that it is the designer's responsibility to ensure that the information and recommendations contained herein are incorporated into the project and that necessary steps are also taken to see that the recommendations are carried out in the field.

The findings in this report are valid as of the present date. However, changes in the subsurface conditions can occur with the passage of time, whether they are due to natural processes or to the works of man, on this or adjacent properties. In addition, changes in applicable or appropriate standards occur, whether they result from legislation or from
the broadening of knowledge. Accordingly, the findings in this report might be invalidated, wholly or partially, by changes outside of our control.

Respectfully submitted,
PARIKH CONSULTANTS, INC.

Very truly yours,
PARIKH CONSULTANTS, INC.

Kandeep Saravanapavan, P.E., G.E. 3040
Project Engineer

Y. David Wang, Ph.D., P.E. 52911
Project Manager
SCCRTC-HIGH WAY 1 AUX LANES (STATE PARK DR TO FREEDOM BLVD)
SANTA CRUZ, CALIFORNIA

Approximate Project Location
Approximate Alignment

NOTE: Zayante-Vergeles (lower) top of rupture plane depth 7km; location derived from Caltrans V2 Fault Database; Holocene age.

Fault data source:

Basemap Sources:
Esri, HERE, Garmin, (c) OpenStreetMap contributors, and the GIS User Community.

Fault map

Project Location

MAP LOCATION

SCC-HIGHWAY AUX LANES (STATE PARK DR TO FREEDOM BLVD)
SANTA CRUZ, CALIFORNIA

JOB NO.: 2020-108-PGR

PLATE NO.: 2
Approximate Project Alignment

DATA SOURCE:

ELEVATION DATA:
USGS NED ned19_n37x00_w122x00_ca_centralcoast_2010 1/9 arc-second 2012 15 x 15 minute IMG

BASEMAP SOURCES: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri, NGCC, (c) OpenStreetMap contributors, and the GIS User Community.

SCC-HIGHWAY AUX LANES (STATE PARK DR TO FREEDOM BLVD)
SANTA CRUZ, CALIFORNIA

JOB NO.: 2020-108-PGR
PLATE NO.: 4
### RECOMMENDED ACCELERATION RESPONSE SPECTRUM
(5% Damping)

**Site Information**

- **Latitude:** 36.9754
- **Longitude:** -121.8905
- **$V_{S30}$ (m/s):** 239
- **Mean Magnitude (for PGA):** 6.97
- **Near Fault Factor,** Derived from USGS Unified Hazard Site
  - **Distance (km):** 13.2

**Recommended Response Spectrum**

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**Source:**

1. Caltrans ARS Online tool (V.3.0.2, https://arsonline.dot.ca.gov/)
2. USGS Unified Hazard Tool (https://earthquake.usgs.gov/hazards/interactive/)
3. Caltrans SDC 2.0 was adopted September 1, 2019. Design Spectrum is based on the USGS 975 year uniform hazard spectrum only.
PROPOSED BORING LOCATION MAP

SCCRTC-HIGHWAY 1 AUX LANES (STATE PARK DR TO FREEDOM BLVD)
APTOS CREEK AND VALENICA CREEK STRUCTURES
SANTA CRUZ, CALIFORNIA
JOB NO.:2021-130-PFR
PLATE NO.: 7C

Legend

- Boring Location

S86a-1
R-21-003
S90-1
S90-2
S90-3
S87-1
R-21-013
R-21-005
R-21-007

Legend

Boring Location

PROPOSED BORING LOCATION MAP

SCCRTC-HIGHWAY 1 AUX LANES (STATE PARK DR TO FREEDOM BLVD)
APTOS CREEK AND VALENICA CREEK STRUCTURES
SANTA CRUZ, CALIFORNIA
JOB NO.:2021-130-PFR
PLATE NO.: 7C

Legend

- Boring Location

S86a-1
R-21-003
S90-1
S90-2
S90-3
S87-1
R-21-013
R-21-005
R-21-007

Legend

Boring Location
APPENDIX II
Classification of rock materials is based on field inspection and is not to be construed to imply mechanical properties.

NOTE: Classification of rock materials is based on field inspection and is not to be construed to imply mechanical properties.
EARTHQUAKE RETROFIT PROJECT 49

NOTE
Classification of each material as shown on this sheet is based upon field inspection and may be considered in applying seismic retrofitting requirements. The terms "lightly graded" and "heavily graded" refer to grading of loose and saturated conditions. The term "lightly compacted" is not to be confused with "compacted".
APPENDIX IV
NO FIELD PHOTOS ARE AVAILABLE AT THIS STAGE
APPENDIX V
STRENGTH PARAMETER AND SHEAR WAVE VELOCITY
EORC-TC-HWY 1 AUK Loren (State Park Dr to Freedom Blvd)

**PROJECT NO.:**
2020-130-PFR

**STRUCTURE:**
State Park Drive OC (As-Built 1961)

**BORING NO.:**
B-1

**BORING ELEV. (ft):**
3.5

**GW DEPTH (ft):**
15

**HAMS ENERGY +:**
60%

**DRILLING RODS (Y/N):**
Y

**Sample No.** | **Layer Thickness (ft)** | **Sample Depth (ft)** | **USCS Type** | **Sample Type** | **Sample Type** | **Unit Weight (pcf)** | **σ_v (psf)** | **σ_v' (psf)** | **C_E** | **C_B** | **C_R** | **C_S** | **F.C.** | **Correlated Strength Parameters** | **Lab Su (psf)** | **Vs (m/s)**
1 | 0 | 8.6 | 5.7 | SW | Q | 12 | SPT | 125 | 656 | 656 | 12.0 | 1.00 | 12.0 | 1.70 | 20.4 | 0.80 | 1.20 | 1.00 | 11.5 | 19.6 | 19.6 | 116
2 | 8.6 | 12.7 | 11.1 | SW | 1 | 44 | SPT | 120 | 1,289 | 1,289 | 44.0 | 1.00 | 44.0 | 1.28 | 56.4 | 0.85 | 1.20 | 1.00 | 44.9 | 57.5 | 42.2 | 116
3 | 12.7 | 18.3 | 15.6 | SP | 1 | 55 | SPT | 120 | 1,829 | 1,792 | 55.0 | 1.00 | 55.0 | 1.09 | 59.8 | 0.95 | 1.20 | 1.00 | 62.7 | 68.1 | 41.6 | 216
4 | 18.3 | 26.8 | 25.6 | SW | 1 | 60 | SPT | 120 | 2,368 | 2,368 | 60.0 | 1.00 | 60.0 | 0.95 | 56.7 | 1.00 | 1.20 | 1.00 | 72.0 | 68.1 | 38.8 | 238

**version 3.4.2 Notes:**
1. The correction factors C_E (Energy Ratio), C_B (Borehole Diameter), C_R (Rod Length) and C_S (Sampling Method-liner), C_N (Overburden) are per Youd 2001.
2. The conversion factors from MC-N to SPT-N are based on Caltrans Geotechnical Manual, “Soil Correlations” module (March 2021): Cohesive: 0.65 Cohesionless: 0.41
3. For fine-grained materials, the correlated shear strengths are based on Caltrans Geotechnical Manual, “Soil Correlations” module (March 2021).
4. The friction angles were estimated based on Caltrans Geotechnical Manual, “Soil Correlations” module (March 2021).
### Soil Parameters & Vs30

**PROJECT NAME:** SCCRTC-HWY 1 Aux Lanes (State Park Dr to Freedom Blvd)  
**PROJECT NO.:** 2020-130-PFR  
**STRUCTURE:** State Park Drive OC (As-Built 1961)  
**BORING NO.:** B-2  
**BORING ELEV. (ft):** 3.5  
**GW DEPTH (ft):** 15  
**BOROHE DIA (in):** 15  
**HAMMER ENERGY:** 60%  
**DRILLING RODS (Y/N):** Y

### Soil Groups and Age Scaling Factor (ASF, Dimensionless)

- **Project No.:** 2020-130-PFR  
- **Cohesionless Materials (SC, SM, SP, SW, GF, & GW, ML):** Holocene  
- **Cohesive Materials (CL, CH, MH, ML, OL, & OH, SC, GC):** Quaternary  
- **Liquefiable Soils (Residual Shear Strength, Sr):** Pleistocene  
- **Young Sedimentary Rocks (Cohesionless):** Holocene  
- **Young Sedimentary Rocks (Cohesive):** Quaternary

### Correlated Strength Parameters

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<th>Sample No.</th>
<th>Layer Thickness (ft)</th>
<th>Sample Depth (ft)</th>
<th>USCS Type</th>
<th>Soil Type</th>
<th>Soil Type (H/D/P)</th>
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<th>$N_{dr}$</th>
<th>$C_r$</th>
<th>$C_s$</th>
<th>$C_f$</th>
<th>$C_o$</th>
<th>$N_{dr}$</th>
<th>$C_f$ Corr.</th>
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<th>$S_r$ (psf)</th>
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</table>

**Version 3.4.2 Notes:**

1. The correction factors $C_r$ (Energy Ratio), $C_s$ (Borehole Diameter), $C_f$ (Red Length) and $C_o$ (Overburden) are per Youd 2001.
2. The conversion factors from MC-N to SPT-N are based on Caltrans Geotechnical Manual, “Soil Correlations” module (March 2021):  
   - Cohesive: 0.65  
   - Cohesionless: 0.41
3. For fine-grained materials, the correlated undrained shear strengths are based on Caltrans Geotechnical Manual, “Soil Correlations” module (March 2021).
4. The friction angles were estimated based on Caltrans Geotechnical Manual, “Soil Correlations” module (March 2021).

---

**Additional Notes:**

- The correction factors $C_r$ (Energy Ratio), $C_s$ (Borehole Diameter), $C_f$ (Red Length) and $C_o$ (Overburden) are per Youd 2001.
- The conversion factors from MC-N to SPT-N are based on Caltrans Geotechnical Manual, “Soil Correlations” module (March 2021):  
  - Cohesive: 0.65  
  - Cohesionless: 0.41
- The friction angles were estimated based on Caltrans Geotechnical Manual, “Soil Correlations” module (March 2021).
- The estimated Vs were correlated based on Caltrans Geotechnical Manual, “Design Acceleration Response Spectrum” module (January 2021).
### Soil Parameters & Vs30

**Project Name:** SCCRTC - HWY 1 Aux Lanes (State Park Dr to Freedom Blvd)

**Date:** 07/15/22

**Structure:**
- Rio Del Mar Blvd (OC)
- Hwy 1 (Q)
- Pleistocene

**Boring No.:** B-2 (6-4-63)

**Depth (ft):** 2.5

**Hamm Energy:** 60%

**Drilling Rods (Y/N):** Y

**Depth (ft):** 15

**Version 3.4.2 Notes:**
1. The correction factors C_E (Energy Ratio), C_B (Borehole Diameter), C_R (Rod Length) and C_S (Sampling Method-Linear), C_N (Overburden) are per Youd 2001
2. The conversion factors from MC-N to SPT-N are based on Caltrans Geotechnical Manual, "Soil Correlations" module (March 2021):
   - Cohesive: 0.65
   - Cohesionless: 0.41
3. For fine-grained materials, the correlated undrained shear strengths are based on Caltrans Geotechnical Manual, "Liquefaction-Induced Lateral Spreading" module (January 2020).
4. The friction angles were estimated based on Caltrans Geotechnical Manual, "Soil Correlations" module (March 2021).
6. The estimated Vs were correlated based on Caltrans Geotechnical Manual, "Design Acceleration Response Spectrum" module (January 2021).

### Soil Groups
- Cohesive Materials (CL, CH, MH, ML, OL, & OH, SC, GC)
- Cohesionless Materials (SC, SM, SP, SW, GP, & GW, ML)
- Residual Soils (Residual Shear Strength, Sr)
- Young Sedimentary Rocks (Cohesionless)
- Young Sedimentary Rocks (Cohesive)

### Age Scaling Factor (ASF, Dimensionless)

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### Correlated Strength Parameters

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**Vs30 (m/s):**
- Hz: 190
- Nq: 65.1
- Ns: 211

**Vs30 (m/s):**
- Coefficients: H: Holocene
- Q: Quaternary
- P: Pleistocene
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<th>Cn</th>
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<td>53.6</td>
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</table>

Version 3.4.2 Notes:
1. The correction factors Cc (Energy Ratio), Cn (Borehole Diameter), Cb (Red Length) and Cs (Sampling Method Factor), Cw (Overburden) are per Youd 2001
2. The conversion factors from MC-N to SPT-N are based on Caltrans Geotechnical Manual, "Soil Correlations" module (March 2021): Cohesive: 0.65 Cohesionless: 0.41
3. For fine-grained materials, the correlated undrained shear strengths are based on Caltrans Geotechnical Manual, "Soil Correlations" module (March 2021).
4. The friction angles were estimated based on Caltrans Geotechnical Manual, "Soil Correlations" module (March 2021).
5. Residual Strength (Sr) is based on Kramer and Wong (2015) as suggested in the Caltrans Geotechnical Manual, "Liquefaction-Induced Lateral Spreading" module (January 2020).
6. The estimated Vs were correlated based on Caltrans Geotechnical Manual, "Design Acceleration Response Spectrum" module (January 2021).
**Soil Parameters & Vs30**

**Project Name:** SCCRTC-HWY 1 AUX Lanes (State Park Dr to Freedom Blvd)

**Project No.:** 2020-130-PFR

**Structure:** 1. Cohesive Materials (CL, CH, ML, MH, OL, & OH, SC, GC)
   2. Cohesionless Materials (SC, SM, SP, SW, GP, & GW, ML)
   3. Liquefiable Soils (Residual Shear Strength, Sr)
   4. Young Sedimentary Rocks (Cohesionless)
   5. Young Sedimentary Rocks (Cohesive)

**Boring No.:** B-3 (8-8-67)

**Boring Elevation (ft):** 4.2

**Hammer Energy:** 60%

**Drilling Rods (Y/N):** Y

**Gw Depth (ft):** 15

**Version 3.4.2 Notes:**
1. The correction factors $C_E$ (Energy Ratio), $C_B$ (Borehole Diameter), $C_R$ (Rod Length) and $C_S$ (Sampling Method-liner), $C_N$ (Overburden) are per Youd 2001.
2. The conversion factors from MC-N to SPT-N are based on Caltrans Geotechnical Manual, "Soil Correlations" module (March 2021):
   - Cohesive: 0.65
   - Cohesionless: 0.41
3. For fine-grained materials, the correlated undrained shear strengths are based on Caltrans Geotechnical Manual, "Soil Correlations" module (March 2021).
4. The friction angles were estimated based on Caltrans Geotechnical Manual, "Soil Correlations" module (March 2021).
5. Residual Strength (Sr) is based on Kramer and Wang (2015) as suggested in the Caltrans Geotechnical Manual, "Liquefaction-Induced Lateral Spreading" module (January 2020).
6. The estimated Vs were correlated based on Caltrans Geotechnical Manual, "Design Acceleration Response Spectrum" module (January 2021).
EVALUATION OF LIQUEFACTION POTENTIAL
LIQUEFACTION ANALYSIS

PROJECT NAME: WSCDC HWY 1, AE2 Lane (State Park Dr to Freedom Blvd)

PROJECT NO.: 2020-130-PFR

STRUCTURE: State Park Drive OC (As-Built 1961)

BORING NO.: B-1

BORING DEPT (ft): 2.5

DESIGN DEPT (ft): 60%

SOIL GROUPS/AGE SCALING FACTOR (ASF, Dimensionless)

<table>
<thead>
<tr>
<th>Soil Group</th>
<th>Age Scaling Factor (ASF, Dimensionless)</th>
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<tr>
<td>Cohesive Materials</td>
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<tr>
<td>Liquefiable Soils</td>
<td>0.60</td>
</tr>
<tr>
<td>Young Sedimentary Rocks</td>
<td>0.90</td>
</tr>
</tbody>
</table>

Hole Depth (ft):

   - Design GW Depth (ft): 15
   - Fault MW = 6.97
   - Maximum LIQ. DEPTH CONSIDERED (ft): 70
3. Liquefiable Soils (Residual Shear Strength, Sr)
   - MSF = 1.21
4. Young Sedimentary Rocks (Cohesionless)
   - Borehole Dia (in): 2.5
   - Hammer Energy: 60%
   - Drilling Rods (Y/N): Y
5. Young Sedimentary Rocks (Cohesive)
   - GW Depth (ft): 15

Sample No. | Upper Thickness (ft) | Sample Depth (ft) | USCS Type | Soil Type | Hole Type | Field Blow Count | Sampler Type | Unit Weight (pcf) | \( \sigma_v \) (psf) | \( \sigma_v' \) (psf) |
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<thead>
<tr>
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<td>656</td>
<td>606</td>
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\[ \text{Sample No} \times \text{Layer Thickness (ft)} \times \text{Sample Depth (ft)} \times \text{USCS Type} \times \text{Soil Type} \times \text{Hole Type} \times \text{Field Blow Count} \times \text{Sampler Type} \times \text{Unit Weight (pcf)} \times \text{\( \sigma_v \) (psf)} \times \text{\( \sigma_v' \) (psf)} \]

Version 3.4.2 Notes:
1. The correction factors \( C_E \) (Energy Ratio), \( C_B \) (Borehole Diameter), \( C_R \) (Rod Length) and \( C_S \) (Sampling Method-liner), \( C_N \) (Overburden) are per Youd 2001
2. The conversion factors from MC-N to SPT-N are based on Caltrans Geotechnical Manual, “Soil Correlations” module (March 2021):
   - Cohesive:
     \[ \text{Cohesive: } 0.65 \]
   - Cohesionless:
     \[ \text{Cohesionless: } 0.41 \]
3. For correction of overburden, \( C_N = \left( \frac{1}{\sigma_v'} \right)^{0.5} \) with a maximum value of 1.7.
4. The influence of Fines Contents are expressed by the following correction: \( (N_1)^{60,CS} = a + b (N_1)^{60} \) where \( a \) and \( b \) = coefficients determined from the following relationships
   - For FC < 5%
     \[ a = 0, \quad b = 1.0 \]
   - For 5% < FC < 35%
     \[ a = \exp(1.76 - \left(\frac{190}{FC^2}\right)), \quad b = \left(0.99 + \left(\frac{FC^{1.5}}{1000}\right)\right) \]
   - For FC > 35%
     \[ a = 5.0, \quad b = 1.2 \]
5. For \( (N_1)^{60} \) greater than 30, clean granular soils are too dense to liquefy and are classed as non-liquefiable.
7. Volumetric Strain of Liquefied Soil is based on Tokimatsu and Seed (1987)

https://parikhnet.sharepoint.com/sites/projects2/Ongoing_Projects/2021/2021-130-PFR/Mark_Thomas_SCCRTC_Hwy_1_State_St_to_Freedom_2020-108_KHA/Calculation/Strength_and_Liq_Ver.3.4.2_8-2-2022

8/2/2022 5:16 PM Ver. 2021-04-07
## LIQUEFACTION ANALYSIS

**PROJECT NO.:** 2020-130-PFR  
**STRUCTURE:** State Park Drive DC (As-Audit 1961)  
**REPORTED BY:** SCCRTC  
**REPORT DATE:** 2.5  
**REPORTED BY:** SCCRTC  
**DEEP DEPTH:** 1.0  
**DEEP DEPTH:** 2.0  

### Liquefaction Analysis

<table>
<thead>
<tr>
<th>Sample No</th>
<th>Layer Thickness (ft)</th>
<th>Sample Depth (ft)</th>
<th>USCS Type</th>
<th>Soil Type</th>
<th>DTH</th>
<th>Field Failure</th>
<th>Sample Type</th>
<th>Unit Weight (pcf)</th>
<th>COHESIONLESS: ( \sigma_v' )</th>
<th>( c )</th>
<th>( \phi_c )</th>
<th>( N_{60,c} )</th>
<th>Corr. C</th>
<th>CRR</th>
<th>CSR</th>
<th>Post-Liq. Settl.</th>
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<td>2.00</td>
<td>21.81</td>
<td>32.44</td>
<td>1.10</td>
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<td>NON-LIQ.</td>
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<td>21.81</td>
<td>32.44</td>
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<td>21.81</td>
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</tbody>
</table>

### Notes
1. The correction factors \( C_1 \) (Energy Ratio), \( C_2 \) (Rod Length), \( C_3 \) (Sampling Method-liner), \( C_4 \) (Overburden) are per Youd 2001
2. The correction factors from MC NCRP SP-015 are based on a G&R correlation method (USGS, "Soil Correlation" dataset March 2021)
3. Cohesive: \( C_4 = 0.45 \) (Calculation)
4. For corrections of overburden, \( C_1 = 3/(5r^{0.5}) \) with a maximum value of 0.27
5. The influence of free contents are represented by the following correction: \( \frac{N_{60,c}}{N_{60}} = 1 + 0.1N_{60} \)
6. Where \( a \) is coefficients determined from the following relationships:
   \[ \frac{N_{60,c}}{N_{60}} = a + b(N_{60}) \]
   \[ a = 0.99 + (0.99 + 0.99) \]
7. For \( r > 30 \), shear strength data are too close to liquefy and are assigned as non-liquefiable
9. Volumetric Strain of liquefied soil is based on Tokimatsu and Seed (1987)

### Project Details
- **PROJECT NO.:** 2020-130-PFR
- **STRUCTURE:** State Park Drive DC (As-Audit 1961)
- **REPORTED BY:** SCCRTC
- **REPORTED BY:** SCCRTC
- **DEEP DEPTH:** 1.0
- **DEEP DEPTH:** 2.0
- **PROJECT NAME:** 2020-130-PFR
- **DATE:** 07/15/22
- **Cohesionless Materials (SC, SM, SP, SW, GP, & GW, ML) H: Holocene
  - Cohesive Materials (CL, CH, ML, MH, OL, & OH, SC, GC)
  - Liquidatable Soils (Residual Shear Strength, Sy)
  - Young Sedimentary Rocks (Cohesive)
  - Young Sedimentary Rocks (Cohesionless)
  - Cohesive (CL, CH, ML, MH, OL, & OH, SC, GC)

### Engineering Notes
- **COHESIONLESS: \( \sigma_v' \)**
- **\( c \)**
- **\( \phi_c \)**
- **\( N_{60,c} \)**
- **Corr. C**
- **CRR**
- **CSR**
- **Post-Liq. Settl.**
LIQUEFACTION ANALYSIS

PROJECT NO.: 2020-130-PFR
PROJECT NAME: SCCRTC Hwy 1, State Park Dr to Freedom Blvd

3. Liquefiable Soil (Relevant Shear Strength, σv)
4. Young Sedimentary Rocks (Cohesionless)
5. Young Sedimentary Rocks (Cohesionless)

1. Cohesionless Materials (SC, SM, SP, SW, GP, GW, ML)
3. Liquefiable Soil (Relevant Shear Strength, σv)
4. Young Sedimentary Rocks (Cohesionless)
5. Young Sedimentary Rocks (Cohesionless)

Cyc. Resistance Ratio (CRR)/Cyc. Stress Ratio (CSR)

For (N1)60,cs greater than 30, clean granular soils are too dense to liquefy and are classed as non-liquefiable.

Version A & B Notes:
1. The correction factors, C, (Energy Ratio), C, (Rod Length), C, (Sampling Method-liner), C, (Overburden) are per Youd 2001
2. The correction factors from MC-N to SPT-N are based on Caltrans Geotechnical Manual, “Soil Correlations” module (March 2021):
3. For correction of overburden, (C, = 1.0)((/1000) with a maximum value of 0.5
4. The influence of fines contents are represented by the following correction: (N1)60,cs = α + β (N1)
   where α and β are coefficients determined from the following relationships
   For (N1)60,cs > 100: α = (0.50 + (100 - (N1)60,cs)) / 100
   For (N1)60,cs < 10: α = (0.50 + (100 - (N1)60,cs)) / 100
   For (N1)60,cs ≥ 10: α ≥ 1.0
5. For (N1)60,cs greater than 10, clean granular soils are too dense to liquefy and are classed as non-liquefiable.

Version 3.4.2 Notes:
1. The correction factors C, (Energy Ratio), C, (Rod Length), C, (Sampling Method-liner), C, (Overburden) are per Youd 2001
2. The conversion factors from MC-N to SPT-N are based on Caltrans Geotechnical Manual, “Soil Correlations” module (March 2021):
3. For correction of overburden, (C, = 1.0)((/1000) with a maximum value of 0.5
4. The influence of fines contents are represented by the following correction: (N1)60,cs = α + β (N1)
   where α and β are coefficients determined from the following relationships
   For (N1)60,cs > 100: α = (0.50 + (100 - (N1)60,cs)) / 100
   For (N1)60,cs < 10: α = (0.50 + (100 - (N1)60,cs)) / 100
   For (N1)60,cs ≥ 10: α ≥ 1.0
5. For (N1)60,cs greater than 10, clean granular soils are too dense to liquefy and are classed as non-liquefiable.

As of 2/22/2022

https://parikhnet.sharepoint.com/sites/projects2/Ongoing_Projects/2021/2021-130-PFR   Mark Thomas SCCRTC Hwy 1 State St to Freedom 2020-108 KHA/Calculation/Strength and Liq_Ver. 3.4.2_8-2-2022
### LIQUEFACTION ANALYSIS

**PROJECT NO.:** SECOR CH-97-1, AUS Lines [State Park Dr to Freedom Blvd]
**PROJECT NO.:** 2020-149NP
**LOCATION:** Hwy Rea Overcrossing
**Borehole No:** B-2 (3-4-80)
**Borehole ID (if any):** 2.5
**Ground Depth:** 25 ft

### Soil Groups
1. Cohesive Materials (SC, SM, SP, SW, GP, & GW, ML)
3. Liquefiable Soils (Residual Shear Strength, Sr)
4. Young Sedimentary Sands (Cohesive)

### Age Scaling Factor (NRF, Dimensionless)
- NRFR:
- Q:

### Volumetric Strain of Liquefied Soil

### Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER Workshops on Liquefaction Resistance of Soils

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Layer Thickness (ft)</th>
<th>Sample Depth (ft)</th>
<th>UHIC Type</th>
<th>UHIC Field Test</th>
<th>Field Test</th>
<th>Test</th>
<th>SPT M (N)</th>
<th>Sample Type</th>
<th>Test</th>
<th>Sample Type</th>
<th>Test</th>
<th>SPT M (N)</th>
<th>60.0CS</th>
<th>Cyc. Resistance Ratio (CRR) / Cyc. Stress Ratio (CSR)</th>
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</thead>
<tbody>
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</table>

### Eqs. Resistance Ratio (EQR)/Eqs. Stress Ratio (ESR)

**Post-Liq. Jenn.

**Version 4.2 Notes:**
1. The correlation factors C_e (Energy Ratio), C_s (Overburden), C_r (C) (Sampling Method-liner), C_f (Overburden) are per Youd 2001
2. The correlation factors from MC-EC (SPT-N) are based on Luttrell (Geotechnical Manual: "Soil Correlations" module (March 2021)
3. Conversions from USCS to CBR are based on Luttrell (Geotechnical Manual: "Soil Correlations" module (March 2021)

### Version 4.2 Notes:
1. The correlation factors C_e (Energy Ratio), C_s (Overburden), C_r (C) (Sampling Method-liner), C_f (Overburden) are per Youd 2001
2. The correlation factors from MC-EC (SPT-N) are based on Luttrell (Geotechnical Manual: "Soil Correlations" module (March 2021)
3. Conversions from USCS to CBR are based on Luttrell (Geotechnical Manual: "Soil Correlations" module (March 2021)
4. For correction of overburden, (S_1 (C/e)^4) with a maximum value of 0.7
5. The influence of fines content are represented by the following correction: (N_60CS = a + b (N_60CS - a = 0, b = 1.0)
6. For correction of fines content: (S_1 (C/e)^4) with a maximum value of 0.7
7. For fines content greater than 30%, clean granular soils are too dense to liquefy and are classed as non-liquefiable.
<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Depth (ft)</th>
<th>Type</th>
<th>UCS Type</th>
<th>Drilling Rods</th>
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The conversion factors from MC-N to SPT-N are based on Caltrans Geotechnical Manual, "Soil Correlations" module (March 2021):

- For overburden, CN = \( \frac{1}{2} \times \text{H/Q/P} \times \text{AS} \times \text{CS} \) for FC > 35% and \( \text{CN} = (1/2) \times A \times B \times C \) for FC ≤ 35%.
- For FC > 5%, \( \text{a} = 1 \).
- For FC ≤ 5%, \( \text{a} = 0 \).

The correction factors, \( C_a \) (Energy Ratio), \( C_b \) (Rod Length) and \( C_r \) (Sampling Method - liner) are based on Takashima and Seed (1979) and Youd, et al., ASCE Journal of Geotechnical and Geoenvironmental Engineering, October 2001, Vol. 127 No. 10.

For cohesionless materials (SC, SM, SP, SW, GP, & GW, ML), H: Holocene, B-3 (8-8-67), SCCRTC-HWY 1 AUX Lanes (State Park Dr to Freedom Blvd).

For cohesive materials (CL, CH, MH, ML, CH, SC, CC), H: Pleistocene, B-3 (8-8-67).

For liquefiable soils (Medium Strength, VS), H: Pleistocene, B-3 (8-8-67).

For young sediments (Rocks), H: Quaternary, B-3 (8-8-67).

Cohesionless: Cohesionless Materials (SC, SM, SP, SW, GP, & GW, ML, H: Holocene, B-3 (8-8-67), SCCRTC-HWY 1 AUX Lanes (State Park Dr to Freedom Blvd).

Cohesive: Cohesive Materials (CL, CH, MH, ML, CH, SC, CC, H: Pleistocene, B-3 (8-8-67)).
Vs30 SPREADSHEET VERIFICATION USING HAND CALCULATION
SOIL PARAMETERS & Vs30

| BOREHOLE (ft) | 6.3 |
| GW DEPTH (ft) | 15 |

**Calc By:**

**Date:**

**PROJECT NAME:** Soil Groups Age Scaling Factor (ASF, Dimensionless)

**PROJECT NO.:**

3. Liquefiable Soils (Residual Shear Strength, Sr) P: Pleistocene
4. Young Sedimentary Rocks (Cohesionless)
5. Young Sedimentary Rocks (Cohesive)

**BOREHOLE DIA (in)=** 4.5

**HAMMER ENERGY =** 60%

**N9D**

\[ v_{sd} \text{ (m/s)} = 160 \]

**GW DEPTH (ft)=** 15

**DRILLING RODS (Y/N)=** Y

\[ v_{s30} \text{ (m/s)} = 186 \]

**ASF**

\[ N = 60 \]

\[ f (°) = 60 \]

\[ Su (psf) = 6.8 \]

\[ Sr (psf) = 0.75 \]

\[ N_1 = 6.1 \]

\[ SPT-N = 28.6 \]

**Note:**

1. The correction factors C_E (Energy Ratio), C_B (Borehole Diameter), C_R (Rod Length) and C_S (Sampling Method-liner), C_N (Overburden) are per Youd 2001.
2. For fine-grained materials, the correlated undrained shear strengths are based on Caltrans Geotechnical Manual, "Soil Correlations" module (March 2021).
3. The friction angles were estimated based on Caltrans Geotechnical Manual, "Soil Correlations" module (March 2021).
4. The conversion factors from MC-N to SPT-N are based on Caltrans Geotechnical Manual, "Soil Correlations" module (March 2021):
   - Cohesive: 0.65
   - Cohesionless: 0.41
5. Residual Strength (Sr) is based on Kramer and Wang (2015) as suggested in the Caltrans Geotechnical Manual, "Liquefaction-Induced Lateral Spreading" module (January 2020).
6. The estimated Vs were correlated based on Caltrans Geotechnical Manual, "Design Acceleration Response Spectrum" module (January 2021).
7. Residual shear strengths were estimated based on the Kramer and Wang (2015) per Caltrans Geotechnical Manual, "Lateral Spreading" module.

**Sample No | Layer Thickness (ft) | Sample Depth (ft) | USCS Type | Soil Type | Field Blow Count | SPT-N eq. | C_E | C_B | C_R | C_S Corr. | C_N | (N_1) | Vs (m/s) | F.Z. | (N_1)भूल | Correlated Strength Parameters | Layer Verified Using Hand Calculation
1 | 3 | 3 | SW | 1 | Q | 4 | SPT | 125 | 315 | 315 | 4.0 | 1.10 | 4.0 | 1.70 | 6.8 | 0.75 | 1.20 | 1.00 | 3.6 | 6.1 | 6.1 | 28.6 | 778
2 | 5 | 5 | SC | 1 | Q | 4 | SPT | 125 | 625 | 625 | 6.0 | 1.00 | 4.0 | 1.70 | 6.8 | 0.75 | 1.20 | 1.00 | 3.6 | 6.1 | 6.1 | 28.6 | 88
3 | 8 | 11 | SW-SM | 1 | Q | 6 | SPT | 125 | 1,250 | 1,250 | 6.0 | 1.00 | 6.0 | 1.30 | 7.8 | 0.85 | 1.20 | 1.00 | 3.6 | 6.1 | 6.1 | 28.6 | 117
4 | 11 | 18 | CL | 2 | Q | 16 | MC | 125 | 2,000 | 2,000 | 10.4 | 1.00 | 10.4 | 0.98 | 10.2 | 0.95 | 1.20 | 1.00 | 11.9 | 11.7 | 11.7 | 117
5 | 18 | 23 | CL | 2 | Q | 16 | MC | 125 | 3,750 | 3,750 | 20.1 | 1.00 | 20.1 | 0.72 | 14.5 | 1.00 | 1.20 | 1.00 | 24.1 | 17.4 | 17.4 | 31.4 | 210
6 | 28 | 33 | CL | 2 | Q | 16 | MC | 125 | 5,000 | 5,000 | 23.0 | 1.00 | 23.0 | 0.97 | 15.4 | 1.00 | 1.20 | 1.00 | 27.6 | 18.5 | 18.5 | 31.7 | 224
7 | 38 | 48 | CL/CH | 2 | Q | 16 | MC | 125 | 6,250 | 6,250 | 23.0 | 1.00 | 23.0 | 0.97 | 15.4 | 1.00 | 1.20 | 1.00 | 27.6 | 18.5 | 18.5 | 31.7 | 224
8 | 48 | 53 | SC | 1 | Q | 23 | SPT | 125 | 7,500 | 7,500 | 23.0 | 1.00 | 23.0 | 0.97 | 15.4 | 1.00 | 1.20 | 1.00 | 27.6 | 18.5 | 18.5 | 31.7 | 224

**Note:**

1. The correction factors C_E (Energy Ratio), C_B (Borehole Diameter), C_R (Rod Length) and C_S (Sampling Method-liner), C_N (Overburden) are per Youd 2001.
2. For fine-grained materials, the correlated undrained shear strengths are based on Caltrans Geotechnical Manual, "Soil Correlations" module (March 2021).
3. The friction angles were estimated based on Caltrans Geotechnical Manual, "Soil Correlations" module (March 2021).
4. The conversion factors from MC-N to SPT-N are based on Caltrans Geotechnical Manual, "Soil Correlations" module (March 2021):
   - Cohesive: 0.65
   - Cohesionless: 0.41
5. Residual Strength (Sr) is based on Kramer and Wang (2015) as suggested in the Caltrans Geotechnical Manual, "Liquefaction-Induced Lateral Spreading" module (January 2020).
6. The estimated Vs were correlated based on Caltrans Geotechnical Manual, "Design Acceleration Response Spectrum" module (January 2021).
7. Residual shear strengths were estimated based on the Kramer and Wang (2015) per Caltrans Geotechnical Manual, "Lateral Spreading" module.
Soil Strength & Liquefaction

Refered Boring B-6 (Elev. 51 ft) in "Iron Horse Regional Trail"

As-Built LOTB for M242-RW1.

**Input Data:**

- **Sample No. 1. Layer thickness from 0 to 5 ft**
- **Sample depth 3 ft.** USCS Type: SM ASF assumed to be Q.
- **Field Blow Count 4** Sampler Type: SPT Hammer Energy: 60%.
- **Unit Weight 125 psf** GW depth: 15 ft - #200 not available.

**Calculation:**

Total Unit Weight \( \bar{\gamma} = 3 \times 125 = 375 \) psf.

Effective Unit Weight \( \bar{\gamma} = \bar{\gamma} = 375 \) psf (GW Depth = 15 ft)

\( SPT - N_{eq, b} = 4 \times 1 = 4 \)

\( CE = \frac{\text{Hammer Energy}}{0.6} = 1.00 \) for 60% hammer energy.

\( N_{60} \) (Correlation) = \( SPT - N_{eq} \cdot CE = 4 \times 1 = 4 \)

\( CB = \begin{cases} 1.0 & \text{for Borehole Diameter} \leq 4.5 \text{ inch (115 mm)} \\ 1.05 & \text{for } 4.5 \leq \text{Borehole Diameter} \leq 6 \text{ inch (150 mm)} \\ 1.15 & \text{for Borehole Diameter} > 6 \text{ inch} \end{cases} \)

\( CB = 1.0 \) since our Borehole Diameter is 4.5 inch.
Soil Strength & Liquefaction

\[
C_s = 1.2 \quad \text{(No liner used)}
\]

\[
C_R = 0.75 \quad \text{(3 + 5 ft (10 ft))}
\]

\[
CN = \left( \frac{Pa}{\nu'o} \right)^{0.5} \quad \text{(Eq. 9 from Youd 2001)}
\]

\[
= \left( \frac{2116.22 \text{ psf}}{375 \text{ psf}} \right)^{0.5}
\]

\[
= 2.38
\]

From Youd et al 2001, \( CN \leq 1.7 \)

\[
CN = 1.7
\]

From Youd et al 2001, Equation 10

\[
CN = 2.2 / \left( 1.2 + \nu'o / Pa \right) = 1.60
\]

\[
N_{bo} = CE \times CB \times CR \times CS \times SPT - Neq = 1 \times 1 \times 0.75 \times 1.2 \times 4 = 3.6
\]

\[
(N_{b})_{bo} = CN : N_{bo} = 1.7 \times 3.6 = 6.12
\]

Sample type: Silty Sand (SM)

Use chart 1 Correlation of SPT \( N_{bo} \) with Friction Angle (after Baule, 1977) in Caltrans Geotechnical Manual — Soil Correlations (March 2021)

\[
\phi \approx 28.2^\circ \quad \text{28.6°}
\]
For shear wave velocity:


\[ V_s = 30 \times (\text{ASF}) \times (N_{60})^{0.23} \times (V_\text{soil})^{0.23} \]

Assume ASF = 1.0 (Quaternary)

\[ V_s = 30 \times 1.0 \times 3.6^{0.23} \times (375 \text{ psf})^{0.23} \]

\[ V_s = 30 \times 1.0 \times 3.6^{0.23} \times (17.955 \text{ kPa})^{0.23} \]

\[ \approx 78 \text{ m/s} \]

If change sample type to GM

\[ V_s = 53 \times (N_{60})^{0.19} \times (V_\text{soil})^{0.18} \approx 113.7 \text{ m/s} \]

\[ V_s = 115 \times (N_{60})^{0.17} \times (V_\text{soil})^{0.12} \approx 202.2 \text{ m/s} \]

If change type to MH

\[ V_s = 26 \times (\text{ASF}) \times (N_{60})^{0.17} \times (V_\text{soil})^{0.32} \approx 81.44 \text{ m/s} \]
LIQUEFACTION ANALYSIS VERIFICATION USING HAND CALCULATION
## LIQUEFACTION ANALYSIS

### PROJECT NAME
- **M-20-101**

### PROJECT NO.
- **NCMSF = 1.26**

### STRUCTURE
- **FAULT MCCRS**

### DESIGN GW DEPTH (ft)
- **0.97**

### MAXIMUM ENERGY =
- **Melrose, NC (0.9)**

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<th>Sample No.</th>
<th>Soil Type</th>
<th>Layer Thickness (ft)</th>
<th>Sample Type</th>
<th>Unit Weight (pcf)</th>
<th>Field Vane Count (pF)</th>
<th>Field Vane Strength (pF)</th>
<th>Unit Vane Count (pV)</th>
<th>Unit Vane Strength (pV)</th>
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<th>CSR</th>
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### Notes:
- 1. The correction factors Cᵥ, Cₛ, and Cₑ (Energy Ratio) are determined using the following relationships:
  - For FC ≤ 0.5
    - Cᵥ = 0.7S
    - Cₛ = 0.05S
    - Cₑ = 0.05S
  - For FC > 0.5
    - Cᵥ = 0.7S
    - Cₛ = 0.05S
    - Cₑ = 0.05S
- 2. The conversion factors from MC-N to SPT-N are based on Caltrans Geotechnical Manual, "Soil Correlations" module (March 2021).
- 3. For correction of overburden, CN = (1/60,CS + 1/60,CS).
- 4. The influence of Fines Contents are expressed by the following correction:
  - For coarse-grained soils (FC ≤ 30): CNC = 1 + (0.05,CS)
  - For fine-grained soils (FC > 30): CNC = 1 + (0.05,CS)
- 5. For (N₄₁₋₃0) (greater than 60), clean granular soils are too dense to liquefy and are classed as non-liquefiable.
- 7. The Soil Groups Age Scaling Factor (ASF, Dimensionless) MAJOR CUT(-)/FILL(+) (ft)
- 8. The conversion factors from MC-N to SPT-N are based on Caltrans Geotechnical Manual, "Soil Correlations" module (March 2021).

### Validation
- **Verified layer Using Hand Calculation**

**References**
Subj:

Liquefaction Hand Calculation

Refereed Boring R-20-B-101 in the LOTB

Input Data:

Sample No. 13  Layer thickness 5  Hole Diameter 4.5 inch.
Sample depth 60 ft.  Soil Type: 1 → Sands, gravels and nonplastic silts.
Field Blow Count 30  Sampler Type: MC  Hammer Energy: 89%
Unit Weight 125 psf  Design GW depth: 4.5 ft  # 100 17%
                   Field GW depth: 9 ft

Calculation:

Total Unit Weight \( W = 60 \times 125 = 7500 \text{ psf} \).
Effective Unit Weight \( W' = 7500 - (60 - 9) \times 62.4 = 4317.6 \text{ psf} \)

\[ \text{SPT \dash Neq_b} = 30 \times 0.41 = 12.3 \quad \text{(Based on field GW)} \]

\[ CE = \frac{\text{Hammer Energy}}{0.6} = 1.483 \approx 1.48 \quad \text{for 89\% hammer energy} \]

\[ \text{N_{60} CE Correlation} = \text{SPT \dash Neq_b} \cdot CE = 12.3 \times 1.483 = 18.24 \approx 18.2 \]

\[ CB = \begin{cases} 1.0 & \text{for Borehole Diameter} \leq 4.5 \text{ inch (115 mm)} \\ 1.05 & \text{for} \ 4.5 \leq \text{Borehole Diameter} \leq 6 \text{ inch (150 mm)} \\ 1.15 & \text{for Borehole Diameter} > 6 \text{ inch} \end{cases} \]

\[ CB = 1.0 \quad \text{since our Borehole Diameter is 4.5 inch.} \]
Liquefaction Hand Calculation

\[ Cs = 1.0 \quad \text{(Standard sampler)} \]

\[ CR = 1.0 \quad \text{\(b_0+5 > 30\)} \]

\[ CN = \left( \frac{Pa}{V_0} \right)^{0.5} \quad \text{(Eq. 9 from Youd 2001)} \]

\[ = \left( \frac{2116.22 \text{ psf}}{4417.6 \text{ psf}} \right)^{0.5} \]

\[ = 0.70 \]

\[ N_{bo} = CE \times CB \times CR \times CS \times SPT - Neg = 1.483 \times 1.0 \times 1.0 \times 1.0 \times 12.3 = 18.24 \approx 18.2 \]

\[ (N_i)_{bo} = CN \cdot N_{bo} = 0.70 \times 18.24 = 12.77 \approx 12.8 \]

Since fine content is available

Equation (6b) & (7b) from Youd 2001.

\[ N_i(b_0)_{cs} = \alpha + \beta (N_i)_{bo} \]

For \(5\% < FC = 17\% < 35\%\):

\[ \alpha = \left( \frac{17b - (190)}{FC^2} \right) = 3.01 \]

\[ \beta = 0.99 + FC^{1.5} / 1000 = 1.06 \]

\[ N_i(b_0)_{cs} = 3.01 + 1.06 \times 12.8 = 16.58 \approx 16.6 \]

\[ CRR_{7.5} = \frac{1}{34 - (N_i)_{bo,cs}} + \frac{N_i(b_0)_{cs}}{135} + \frac{50}{(10 \cdot N_i(b_0)_{cs} + 45)^2} - \frac{1}{200} \]

\[ = 0.176 \quad \text{(Equation 4 in Youd 2001)} \]

\[ \approx 0.18 \]
Liquefaction Hand Calculation

Design Groundwater depth : 4.5 ft

\[ \sigma' = 7500 - (60 - 4.5) \times 62.4 = 4036.8 \text{ psf} \]  
(Based on design 6w)

Stress Reduction Coefficient \( (r_d) \) Equation 3 from yourd 2001

\[ r_d = \frac{1.000 - 0.4113 z^{0.5} + 0.04052 z + 0.001753 z^{1.5}}{1.000 - 0.4177 z^{0.5} + 0.05729 z^2 - 0.006205 z^{1.5} + 0.001210 z^3} \]

where \( z = \) depth beneath ground surface in meters.

\[ z = 60 \text{ ft} = 18.28 \text{ m} \]

\[ r_d = 0.659 \approx 0.7 \]

CSR : Equation (5) from yourd 2001

\[ \text{CSR} = 0.65 \times A_{max} \left( \frac{\sigma'_{max}}{\sigma'_{o}} \right) r_d \]

\[ = 0.65 \times 0.97 \times \left( \frac{7500}{4036.8} \right) \times 0.659 \]

\[ = 0.77 \approx 0.8 \]

Relative Density

\[ D_r = 0.36 \times \left( \frac{N_{bo} - C_e \text{ cor.}}{\sigma'_{o} + 1.5} \right)^{0.37} = 0.36 \times \left( \frac{18.2}{4036.8 + 1.5} \right)^{0.37} \]

\[ = 0.669 \approx 0.70 \]
Based on Hynes and Olsen (1999):

\[ f = \begin{cases} 
0.8, & \text{for relatively loose deposit} \\
0.7, & \text{for medium dense} \\
0.6, & \text{for dense or slightly overconsolidated deposits}
\end{cases} \]

For very dense or higher overconsolidation (stress history and aging effects), the exponent \( f \) may be less than 0.6.

In our spreadsheet, using Dr to determine the density,

\[ f = \begin{cases} 
0.8, & \text{if } Dr \leq 40\% \\
0.8 - (Dr - 0.4)/2, & \text{if } 40\% < Dr \leq 60\% \\
0.7 - (Dr - 0.6)/2, & \text{if } 60\% < Dr \leq 80\% \\
0.6, & \text{if } Dr > 80\%
\end{cases} \]

\( f \) is on conservative side compared to Hynes and Olsen (1999).

Since \( Dr = 0.669 = 66.9\% \)

\[ f = 0.7 - (0.669 - 0.6)/2 = 0.666 \approx 0.7 \]

Based on Hynes and Olsen (1999) or Eq. 31 from Youd 2001.

\[ K_{Df} = \left( \frac{\sigma'}{\sigma} \right)^{-1} = \left( \frac{4317.6}{2116.22} \right)^{0.666 - 1} = 0.788 \approx 0.8 \]

(Based on field Gw = 9 ft, \( \sigma' = 4317.6 \) psi)
Level Ground $k_\alpha = 1.0$

$M_w = 6.85 \leq 7.5$

Magnitude Scaling Factor:

$$MSF = \frac{10^{2.24}}{M^{2.56}} = \frac{10^{2.24}}{6.85^{2.56}} = 1.26$$

Factor of Safety against Liquefaction:

$$FS = \frac{CRR_{75}}{CSR} \times MSF \times k_0 \times k_\alpha$$

$$= \frac{0.176}{0.77} \times 1.26 \times 0.788 \times 1.0$$

$$= 0.227$$

$\approx 0.23$
APPENDIX VI
NO EXCEPTIONS TO POLICY
APPENDIX VII