Appendix H. Simi Valley Double Track and Platform Project Preliminary Geotechnical Design Report

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CTO 48 SCORE Ventura Corridor
Segment 1 - Simi Valley Double Track and Platform
Preliminary Geotechnical Design Report
SIMI VALLEY, CALIFORNIA

January 2020

Prepared for:

Southern California Regional Rail Authority (SCRRA)
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Prepared by:

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January 31, 2020

METROLINK
Mr. Colm McKenna
270 E. Bonita Ave
Pomona, CA 91767

Attn: Mr. Colm McKenna, Senior Railroad Civil Engineer

Dear Mr. McKenna,

We are pleased to present this preliminary geotechnical design report summarizing the results of our geotechnical study for the proposed *CTO 48 SCORE Ventura Corridor, Segment 1 - Simi Valley Double Track and Platform* project located at Simi Valley, California. This report summarizes the work performed, data acquired, and our preliminary findings, evaluations, and recommendations for preliminary design of the project.

We appreciate the opportunity to provide geotechnical services on this project and trust the information in this report meets the current project needs. If you have any questions regarding this report or if we may be of further assistance, please contact the undersigned.

Respectfully submitted,

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1.0 INTRODUCTION

1.1 PROJECT DESCRIPTION

The Simi Valley Double Track and Platform Project (Project) is the first segment of Southern California Regional Rail Authority's (SCRRA's) Southern California Optimized Rail Expansion (SCORE) Program which implements rail infrastructure improvements necessary to support expanded Metrolink commuter rail passenger services. Key Project improvements include:

- Adding approximately 2.15 miles of new track between new Control Point (CP) Sequoia and new CP Arroyo;
- Improving grade crossings, such as new track panels and warning devices at Tapo Canyon Road, Tapo Street, E. Los Angeles Avenue, and Hidden Ranch Drive; and
- Adding a second platform at Simi Valley Station with a pedestrian underpass crossing.

Additionally, a concrete retaining wall consisting of two interlocked Enviroblocks or similar gravity block wall is proposed to the south of the existing rail track between the Simi Valley Station and Ralston Street.

The addition of siding track will provide additional capacity for train operations between the new CP Sequoia and CP Arroyo. This siding extension configuration will improve the safety and reliability of the commuter rail system on the Ventura Corridor. The addition of the siding platform at the Simi Valley Station also provides flexibility for trains entering and leaving the station. Based on discussions with the design team, the proposed pedestrian underpass will consist of a precast concrete box type structure (approximately 13 feet by 13 feet). Proposed improvements are shown on Site maps in Appendix D.

1.2 PROJECT LOCATION

The Project is located in the City of Simi Valley, Ventura County, California. The Project improvements are proposed within the SCRRA right of way (ROW) from Control Point (CP) Strathearn, Milepost (MP) 432.8 to CP Davis, MP 440.8 (Site). A Site vicinity map is presented in Figure 1 in Appendix A.

1.3 PURPOSE AND SCOPE

The purpose of this preliminary geotechnical design report is to collect subsurface information at the Site and provide preliminary geotechnical recommendations for the preliminary design of the proposed Project. The scope of work for this preliminary geotechnical design report included the following tasks:

- Review geotechnical maps and reports available online or in our in-house library that are relevant to the Site.
- Perform a Site reconnaissance to mark the proposed boring locations and contact Underground Service Alert (USA, also known as DigAlert) for utility clearance. Perform a geophysical survey to identify potential buried utilities and other detectable subsurface obstructions in the immediate vicinity of proposed boring locations prior to performing field exploration.

- Perform a subsurface exploration consisting of drilling, logging, and sampling of six (6) hollow-stem auger (HSA) borings to depths ranging between 20 and 50 feet below ground surface (bgs) (see Section 2.1 and Figures 2 to 4 in Appendix A).
- Perform geotechnical laboratory testing on selected soil sampling.
- Perform geotechnical evaluation of the collected data.
- Prepare this preliminary geotechnical design report presenting our preliminary findings and geotechnical recommendations for the proposed improvements.

2.0 GEOTECHNICAL INVESTIGATION AND LABORATORY TESTING

2.1 Subsurface Exploration

Subsurface exploration consisted of advancing six (6) 8-inch diameter HSA borings to a maximum depth of approximately 50 feet bgs. The borings are located along an approximately 2-mile stretch of the Ventura Subdivision from CP Sequoia to CP Arroyo. The approximate locations of the borings are shown in Figures 2 to 4 in Appendix A. Approximate boring coordinates, ground surface elevations, and depths explored are summarized in Table 2-1.

Table 2-1. Subsurface Exploration Information

Boring ID	Latitude	Longitude	Ground Surface Elevation (feet)	Exploration Depth (feet)
A-19-001	34.27204	-118.72361	950	21.5
A-19-002	34.27203	-118.71703	958	14.5
A-19-003	34.27200	-118.70925	968	21.5
A-19-004	34.27206	-118.70185	981	21.5
A-19-005	34.26971	-118.69441	986	50.0
A-19-006	34.26937	-118.69381	988	21.5

Note:
(1) Information presented in this table is approximate.
(2) Ground surface elevations were obtained from Google Earth Pro™.

HDR conducted a Site reconnaissance on October 22, 2019 to evaluate the surface conditions and accessibility of the Site for field equipment and to mark the proposed boring locations. The borings were marked in the field by measuring the distance from existing Site features and by using a global positioning system (GPS). Subsequently, Underground Services Alert of Southern California (also known as DigAlert), SCRAA Communication and Signals, and Union Pacific Railroad (UPRR) were contacted to identify subsurface utilities and obtain clearance for advancing borings at the Site. Additionally, an independent third-party geophysical subconsultant (Southwest Geophysics, Inc.) was used by HDR to clear the boring locations prior to drilling. Southwest Geophysics completed utility clearance on October 25, 2019.

Borings were drilled on November 20 and 21, 2019 using a CME-75 drilling rig equipped with an 8-inch diameter HSA. Standard Penetration Tests (SPTs) were performed using a SPT sampler driven for a total penetration of 18 inches (or until practical refusal) into soil at 5-foot intervals within the HSA borings. The sampler was driven using a 140-pound automatic hammer falling from a 30-inch height and the blow counts per 6 inches of penetration were recorded in the boring logs. The total number of hammer blows required to drive the sampler the final 12 inches is termed the SPT blow count. At select depths within the HSA borings, ring samples were collected using a Modified California (MC) sampler. The field sampling procedures were conducted in accordance with ASTM Standard Test Methods D1586 and D3550 for SPT and split-barrel sampling of soil, respectively. In addition to driven samples, bulk samples were also collected from drill cuttings at selected borings.

The test borings were logged in the field by an HDR geotechnical staff. Each soil sample collected was reviewed and described in general accordance with the Unified Soil Classification System (ASTM D2487). Soil samples were delivered to *AP Engineering and Testing* for laboratory testing. Soil corrosivity screening was performed on select samples by HDR's internal laboratory. After completion of drilling, the borings were backfilled with soil cuttings. Borings that encountered groundwater were backfilled with cement-bentonite grout. Geotechnical boring logs are included in Appendix B. Note that the blow counts presented on the logs are actual field blow counts and have not been adjusted for the effects of overburden pressure, input driving energy, rod length, sampler correction, boring diameter, or other factors.

2.2 GEOTECHNICAL LABORATORY TESTING

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

- In-situ moisture content and density;
- Atterberg limits;
- Grain-size distribution;
- Percent passing No. 200 sieve;
- Laboratory Compaction (maximum dry density and optimum moisture content);
- Sand Equivalent;
- Direct Shear;
- Consolidation;
- Expansion Index;
- R-Value; and
- Corrosivity (soluble sulfate contents, chloride, pH, and resistivity).

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

3.0 GEOLOGY AND FAULTING

3.1 REGIONAL GEOLOGY

The Site is located within the Ventura Basin of the Transverse Ranges geomorphic province of California. The Transverse Ranges are characterized by an east-west trend consisting of a complex group of mountain ranges and valleys, extending over 320 miles from the Mojave and Colorado Desert Provinces to Point Arguello at the Pacific Ocean. The mountain ranges include the Santa Susana Mountains to the north, Simi Hills to the south and east, and to the west unnamed hills that separate the Simi Valley from Tierra Rejada Valley and Little Simi Valley. Late Cretaceous to late Tertiary marine sedimentary units, along with minor late Cenozoic nonmarine fluviatile sedimentary deposits, are exposed over most of the upland terrain. Quaternary alluvial sediments derived from erosion of the surrounding hills and mountains filled the valley and canyon bottoms throughout the Ventura Basin (California Division of Mines and Geology [CDMG], 1997).

3.2 SITE GEOLOGY

The Site is generally located on a surficial deposit denoted as Young Alluvial Fan Deposits (Qyf) (California Geologic Survey [CGS], 2012). This deposit is described as unconsolidated to slightly consolidated, undissected to slightly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon.

According to CGS (2012), a section of the Site along the E. Los Angeles Avenue and the east of the unlined channel is located on geological unit denoted as Alluvial Wash Deposits (Qw) which is described as unconsolidated sandy and gravelly sediments deposited in recently active channels of streams and rivers. This geologic unit may contain loose to moderately loose sand and silty sand.

Surficial soils may also contain artificial fill and other materials from previous construction activity at the Site. A Site geologic map is presented in Figure 5 in Appendix A.

Our geotechnical investigations indicated that general subsurface soil conditions at the Site are as follows:

- Vicinity and West of Tapo Canyon Road: Low plasticity silt overlaying silty sand, clayey sand and clay;
- Vicinity of Tapo Street and Angus Avenue: Silty Sand; and
- Vicinity of Simi Valley Station: Lean clay overlaying clayey sand and fat clay.

Detailed subsurface soil conditions are presented on the boring logs in Appendix B.

3.3 FAULTING AND SEISMICITY

Our review of California Earthquake Hazards Zone Application (EQ Zapp) available online by California Geological Survey (CGS, 2019) and the USGS Quaternary Fault and Fold Database of the United States (USGS, 2019a) indicates that the Project site is not underlain by known active or potentially active faults, nor does the Site lie within an Alquist-Priolo Earthquake Fault Zone.

The principal seismic hazard that could affect the Site is ground shaking resulting from an earthquake occurring along one of several major active or potentially active faults in Southern California. Table 3-1 provides relevant fault parameters for faults (sorted based on distance to the

Site) located within a 25 mile radius from the Project site. A regional fault map is provided in Figure 6 in Appendix A.

Table 3-1. Nearby Faults

Fault Name	R_{rup} (1) (km)	Maximum Moment Magnitude (M_W)	Fault Type
Chatsworth Fault	2.7	6.4	Reverse
Simi-Santa Rosa Fault Zone (Simi-Santa Rose Section)	3.2	6.8	Strike-Slip
Sierra Madre Fault Zone (Santa Susana Section)	8.6	6.8	Reverse
Northridge Hills	10.5	6.4	Reverse
Oak Ridge (Onshore)	13.2	7.4	Reverse
Northridge	15.1	6.8	Reverse
Holser alt 1	15.8	6.7	Reverse
Simi-Santa Rosa fault zone (Camarillo-Santa Rosa Section)	16.9	6.8	Strike-Slip
San Cayetano	18.3	7.2	Reverse
Anacapa-Dume (alt 1)	22.4	7.2	Reverse

Notes:

1. R_{rup} = Closest distance from Boring A-19-003 to fault rupture plane based on Caltrans (2019); Km = Kilometer

4.0 SITE CONDITIONS

4.1 Existing Surface Conditions

Existing surface conditions at the Site include a passenger rail station (Simi Valley Station), boarding platform, single rail track, and a paved parking lot located north of the existing platform. Numerous residential and commercial development, street pavement, and different forms of vegetation (from grass and small bushes to large trees) exist in the vicinity of the Site. A side rail track extends parallel to the main rail track on the north side between Argus Avenue and Tapo Canyon Road. An unlined channel intersects with the rail track to the west of Ralston Street, connects to a culvert under E. Los Angeles Avenue and then continues south of the rail track. The southern portion of the channel is lined.

The existing ground surface elevation at the Site ranges from approximately 945 to 988 feet North American Vertical Datum 88 (NAVD 88). In general, surface water appears to drain to the west of the Site.

4.2 Subsurface Earth Materials

Subsurface materials encountered along the Project alignment generally consist of artificial fill ranging from about zero (not encountered) to 9 feet in thickness underlain by alluvial deposits.

The artificial fill is likely associated with the construction of the existing railroad tracks, rail station, and existing road crossings. In general, the fill material consists of loose to medium dense poorly-graded sand with silt, silty sand, clayey sand, and clays, and contains varying amounts of fine gravel.

The alluvial deposits encountered along the Project alignment extend from the ground surface or the bottom of the fill layer to the maximum depth explored (approximately 50 feet bgs). The alluvial deposits encountered within the upper 20 feet of Borings A-19-001 to A-19-004 varies from loose to stiff sandy silt to silty sands with relative densities ranging from loose to medium dense. Some clayey sands and lean clays were also encountered at depths below 15 feet bgs.

Adjacent to the existing Simi Valley Station (Borings A-19-005 and A-19-006), alluvial deposits consist of lean clay, sandy lean clay, and fat clay ranging in consistencies from soft to stiff between depths of 5 and 39 feet bgs. At depths greater than 39 feet bgs in Boring A-19-005, alluvial deposits consist of medium dense sand with silt.

4.3 Engineering Properties of Subsurface Materials

Engineering properties of the subsurface materials were developed based on the results of our geotechnical field and laboratory testing. Laboratory test results are presented in Appendix C and are briefly discussed below.

4.3.1 Shear Strength

Based on the direct shear test results, the cohesion intercept (c) and friction angle (ϕ) representing the effective ultimate shear strength of the tested soils ranged from about 100 pounds per square foot (psf) to 250 psf and 20 to 32 degrees, respectively. Based on the laboratory test results, SPT blow counts, and soil types, generalized shear strength parameters and unit weights selected for design are presented in Table 4-1 and grouped based on soil type. The test results are presented in Table C-1.

Table 4-1. Preliminary Soil Design Parameters

Generalized Soil Type	Depth Below Grade (feet)	Total Unit Weight (pcf)	Friction Angle (degrees) (1)	Cohesion (psf) (1)
Silty or Clayey Sand	0-20	120	30	-
Silty or Clayey Sand (2)	0-5	120	30	-
Lean Clay (2)	5-39	120	25	150
Sand with Silt (2)	39-50	120	32	-

Note:

(1) Ultimate shear strength parameters based on SPT blow counts (NAFVAC, 1986) and laboratory test results.

(2) For soils located at Simi Valley Station

4.3.2 Density and Compaction

The measured dry density in the upper 5 feet of subgrade soils ranged between approximately 80 pcf and 111 pcf with an average of 98 pcf. The water content of the onsite soils in the upper 5 feet varied between approximately 4 and 21 percent with an average of approximately 13 percent.

Using the laboratory maximum dry density values obtained based on the ASTM Test Method ASTM D1557, the estimated maximum dry density of the existing near-surface subgrade materials (upper 5 feet) ranges from approximately 114 to 130 pcf with an average of 120 pcf. The optimum moisture content ranged from about 8 to 15 percent with an average of 12 percent. Therefore, the onsite soils tested in the upper 5 feet are estimated to have been at about 82 percent relative compaction, and about 1 percent above optimum moisture content on average.

4.3.3 Expansive Soils

Expansion index (EI) testing was conducted at two locations. The EI test represents the tendency of soils to expand when wetted or contract when dried. Test results indicated that the soil within the upper 5 ft of borings A-19-003 and A-19-005 had EI values of 2 and 109 corresponding to very low and high expansion potential, respectively. In addition, laboratory tests performed on the near-surface soil samples (upper five feet) adjacent to the Simi Valley Station indicated liquid limits of 27 and 36 and plasticity indices of 10 and 16. Based on the test results, the likelihood of encountering expansive soils adjacent to the Simi Valley Station is considered high.

4.3.4 Corrosion Potential

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

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

Based on the corrosion test results in Table 4-2 and corrosion potential based on various guidelines presented in Table 4-3, the subsurface soils at the Site have a low corrosion potential to buried concrete materials and are generally considered corrosive to buried ferrous metals.

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

Table 4-2. Summary of Corrosion Test Results

Boring ID	Sample Depth (feet)	pH	Minimum Resistivity (ohm-cm)	Sulfates (ppm)	Chlorides (ppm)
A-19-001	3	7.4	1,680	47	0.7
A-19-005	3-5	7.4	1,560	46	3.1
A-19-005	15-20	7.6	1,440	95	3.6

Note:

ft= feet; ohm-cm = ohm centimeters; ppm = parts per million

Table 4-3. Summary of Corrosion Potential

Boring ID	Sample Depth (feet)	Caltrans Corrosion Criteria (1)	NACE Corrosion Potential (2)	Sulfate Attack Potential (3)
A-19-001	3	Not Corrosive	Corrosive	Negligible
A-19-005	3-5	Not Corrosive	Corrosive	Negligible
A-19-005	15-20	Not Corrosive	Corrosive	Negligible

Notes:

(1) Corrosivity screening established using the Caltrans Corrosion Guidelines (2018).

(2) Corrosivity screening established using the National Association of Corrosion Engineers (1984).

(3) Corrosivity screening established using Portland Cement Association (1988).

4.3.5 Compressible Soils

Our review of the consolidation test results indicate that the clay layers encountered within the upper 50 feet are considered to be moderately compressible when subjected to additional loads such as moderate to heavy foundation loads and/or additional fill soils.

4.4 GROUNDWATER

During our field exploration, groundwater was not encountered in Borings A-19-001 through A-19-004 to the maximum depth explored of 20 feet bgs. Groundwater was encountered at depths of approximately 22 and 20 feet bgs in Borings A-19-005 and A-19-006, respectively.

A preliminary review of the California Department of Water Resources (CDWR, 2019) available groundwater well information indicates that there is a monitoring well (State Well Number: 02N17W07J005S) within 700 feet from the Simi Valley Station. Depths to groundwater in this well ranges from about 21 to 27 feet bgs between the years of 1998 and 2005. Additionally, well No. 02N18W12B001S is located approximately 2,500 feet north of Boring A-19-002 with groundwater depth of approximately 98 feet bgs between the years of 1989 and 1991.

A review of the California Division of Mines and Geology (CDMG, 1997) Historically Highest Groundwater Contours Map shows that near the Simi Valley Station, historically highest groundwater levels ranged between 15 and 20 feet bgs. For areas along the Project alignment located west of the existing station, historically highest groundwater levels are indicated to be greater than 50 feet bgs.

A historically high groundwater depth of 15 feet bgs was used in our engineering analyses. Fluctuations of the groundwater level, localized zones of perched water, and an increase in soil moisture should be anticipated during and following the rainy seasons or periods of locally intense rainfall or storm water runoff.

5.0 GEOLOGIC AND SEISMIC HAZARDS

5.1 SEISMIC HAZARDS

5.1.1 Fault Rupture

As mentioned in Section 3.3, no active or potentially active faults are known to traverse the Site, and the Site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone. The nearest Alquist-Priolo Special Study Zone is approximately 1.8 miles north of the Site. The nearest Special Studies Zones is shown on Seismic Hazard Map presented in Figure 7 in Appendix A.

5.1.2 Liquefaction

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

Portions of the site are located within an area designated as potentially liquefiable by the California Geological Survey (2019). Liquefaction analysis was performed on Boring A-19-005 located near the proposed pedestrian underpass. Our preliminary liquefaction evaluation was conducted using a peak horizontal ground acceleration of 0.37g (AREMA Level II seismic event) weighted for a Moment Magnitude (Mw) of 6.7 and a design groundwater level of 15 feet bgs. Based on this analysis, granular subsurface soils between approximate depths of 24 and 29 feet, and between 40 and 50 feet bgs are susceptible to liquefaction.

5.1.3 Seismically-Induced Settlements

Seismically-induced settlements consist of dry dynamic settlement (above groundwater) and liquefaction-induced settlement (below groundwater). This settlement occurs primarily within loose to moderately dense sandy soils due to a reduction in volume during and shortly after an earthquake event. Dry dynamic settlement is considered relatively small due to the presence of high groundwater table. The liquefaction-induced settlement was estimated to be about 3.5 inches for the area near the proposed pedestrian underpass. If the estimated total settlements (static and seismic) are in excess of the tolerable settlement for the pedestrian underpass structure, the liquefaction potential should be mitigated. Alternatives for liquefaction mitigation may include ground improvement (i.e., vibro-replacement dry stone columns, compaction grouting, deep soil mixing) or using a deep foundation system that extends below the bottom of the liquefiable layer. Following a significant seismic event, areas without ground improvement or not supported on a pile foundation system (i.e. track bed, platforms, etc.) may be affected by liquefaction-induced settlements. For these locations, track inspections will need to be followed and maintenance repair and minor re-leveling of the track bed may be required.

5.1.4 Lateral Spreading

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

5.1.5 Seiches and Tsunamis

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

5.1.6 Earthquake-Induced Flooding

Earthquake-induced flooding is caused by dam failures or other water-retaining structure failures as a result of seismic shaking. A review of the 2015 Ventura County Multi-Hazard Mitigation Plan, Dam Inundation Areas (Ventura County, 2015), indicates that the Site is located within an area susceptible to dam inundation due to the failure of the Las Llajas Canyon Dam. Therefore, the risk related to earthquake-induced flooding exists at the Site.

5.2 FLOODING

According to the Federal Emergency Management Agency (FEMA) Flood Insurance Rate Map Number 06111C0864E (FEMA, 2019), the majority of the Site is located within Zones AE, AH, and AO which are determined to be Special Flood Hazard Areas or areas subject to flooding by the 1% annual chance flood (100-year flood). Therefore, the risk related to natural flooding exists at the Site.

5.3 LANDSLIDES

The Site is located in a relatively flat terrain. Additionally, the area is not mapped by CGS (2019) within a landslide zone as shown on the Seismic Hazard Map in Figure 7 in Appendix A. Therefore, the risk of landslides at the Site is considered low.

5.4 EXPANSIVE/COLLAPSIBLE SOILS

Soil expansion describes the tendency of the soil to expand when wet or contract when dried. Soil collapse indicates the tendency for soil to contract suddenly when loaded and wetted. Although the immediate area is not known to contain soils exhibiting these behaviors, testing of samples obtained in the vicinity of Simi Valley Station indicated that expansive soils should be expected at this location.

5.5 SUBSIDENCE

Subsidence is the sinking of the ground surface caused by the compression of earth materials or the loss of subsurface soil due to underground mining, tunneling, erosion, or pumping/extraction of groundwater. The major causes of subsidence include fluid withdrawal from the ground, decomposing organics, underground mining or tunneling, and placing large fills over compressible earth materials. The effective stress on underlying soils is increased resulting in consolidation and settlement. Subsidence may also be caused by tectonic processes. The Site is not located in an area of known ground subsidence or within any delineated zones of subsidence due to groundwater pumping or oil extraction (USGS, 2019c). Accordingly, the potential for subsidence to occur at the Site is low.

6.0 PRELIMINARY GEOTECHNICAL RECOMMENDATIONS

6.1 FOUNDATIONS AND BEARING CAPACITY

The selection of an appropriate foundation system should consider the soil conditions, anticipated structural loads, tolerable settlements, cost of liquefaction mitigation, cost of structure replacement, and impact of service interruption.

Based on these considerations, the proposed pedestrian underpass may be supported on a mat foundation underlain by geogrid-reinforced crushed rock. It is important to note that supporting the proposed pedestrian underpass on a mat foundation will not mitigate the effects of liquefaction at the Site. Following a significant seismic event, liquefaction-induced settlements may occur and the proposed structure may need to be repaired or replaced. Alternatively, if liquefaction-induced settlements are not tolerable, the pedestrian underpass can be supported either on mat foundation founded on improved ground (i.e., vibro-replacement dry stone columns, compaction grouting, deep soil mixing) or on a deep foundation system. Foundation recommendations for a mat foundation supported on a geogrid-reinforced crushed rock are provided in Section 6.1.1.

Minor uninhabited structures including station platform, at-grade crossings, equipment pads, and other miscellaneous structures may be supported on spread footings founded on engineered fill. The use of geogrid reinforced engineered fill is optional for these improvements since these are considered minor structures. Due to the potential for liquefaction and after a strong seismic event, the serviceability of these structures may not be acceptable and may require repair.

6.1.1 Pedestrian Underpass Bearing Capacity

We understand that the pedestrian underpass consists of a precast concrete box structure which will be placed at the bottom of the excavation at an approximate depth of about 13 feet below the existing grade. For preliminary foundation design purpose, an allowable net bearing capacity of 2,500 psf may be used for mat foundation design with a minimum footing width of 10 feet provided that the foundations are constructed in accordance with the recommendations of Section 6.3 including overexcavation and the use of geogrid and crushed rock. These values may be increased by one-third when considering loads of short duration, such as those imposed by wind or seismic forces.

6.1.2 Minor Structures Bearing Capacity

For minor uninhabited structures supported on properly compacted subgrade, an allowable bearing capacity of 1,500 pounds per square foot (psf) may be used with a minimum embedment of 24 inches below the lowest adjacent grade, and minimum footing width of 18 inches. This allowable bearing pressure may be increased by 250 psf for each additional foot of embedment, to a maximum value of 2,000 psf. This value may be increased by one-third when considering loads of short duration, such as those imposed by wind and seismic forces. The footing dimension and reinforcement should be designed by the Project structural engineer. A coefficient of resistance of 0.3 for lateral sliding resistance may be assumed in the preliminary designs.

6.2 SEISMICITY AND PRELIMINARY SEISMIC DESIGN CRITERIA

Based on our evaluation of the subsurface conditions at Simi Valley Station, the Site is categorized as Site Class E. Site Class E was conservatively assumed for the other locations along the Project alignment for the preliminary design. This assumption may be updated in future design stages.

A preliminary seismic hazard analysis was performed using the USGS Unified Hazard Tool (USGS, 2019b) to evaluate anticipated ground motions at the Site. Peak ground accelerations (PGAs) were estimated for upper bound return periods for the three seismic levels recommended in the American Railway Engineering and Maintenance-of-Way Association manual (AREMA, 2019). These seismic events include Level I (50 to 100-year return period), Level II (200 to 475-year return period), and Level III (1,000 to 2,475-year return period). PGAs for each return period were initially estimated for Site Class B and were then adjusted to Site Class E. Table 6-1 presents the results of our preliminary seismic analysis. During the final design stage, the return period corresponding to each seismic event should be adjusted using the AREMA risk factors and an acceleration response spectrum (ARS) should be developed for each seismic event in accordance with Chapter 9 of AREMA (2019).

Table 6-1. Preliminary Peak Horizontal Ground Accelerations

Seismic Event Level	Return Period (years)	Peak Horizontal Accelerations(1), (g)
I	100	0.33
II	475	0.37
III	2,475	0.63

Note:

1) g = unit of gravitational acceleration.
2) USGS (2019b) for Site Class B using Dynamic Conterminous 2014 dataset (V4.1.1). Acceleration Response spectra are adjusted to the Project site class (Site Class E) from baseline Site Class B data per AREMA (2019).

6.3 EARTHWORK

6.3.1 Site Preparation

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

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

6.3.2 Overexcavation

Pedestrian Underpass: For construction using a geogrid mat rather than ground improvement, the area intended for the pedestrian underpass mat foundation should be overexcavated a minimum of 3 feet below the bottom of the proposed footing grade, and replaced with geogrid-reinforced crushed rock to limit effects of liquefaction-induced differential settlement on the underpass structure and provide a stable working platform. Tensar TriAx TX5 geogrid (or equivalent) should be placed as recommended by Tensar in their Installation Guide (Tensar, 2019).

Lateral limits of overexcavation should be established at a minimum distance of 3 feet horizontally beyond the pedestrian underpass footprint. The exact extent of removals can best be determined during excavation when direct observation and evaluation of subsurface materials are possible.

If ground improvement is performed, the upper few feet of the existing soils will be disturbed and some remedial grading will be required. In addition, there may be bulking of the upper soils from the ground improvement process. We recommend that the improvement area be overexcavated to a depth of at least one foot below the bottom of the footings. Depending on the amount of disturbance, the overexcavation may have to be deepened. This overexcavation should extend the full width of the improved area or at least of 3 feet outside the underpass foundation, whichever is greater.

At- Grade Station Platform: To provide a firm and uniform support for the proposed platform and to reduce the potential total and differential settlements, the platform area in general should have, at a minimum, 2 feet of engineered fill underneath the finished pad grade. The lateral limits of overexcavation and engineered fill should be established at a minimum distance of 2 feet horizontally beyond the edge of the platform. The exact extent of removals can best be determined during grading when direct observation and evaluation of subsurface materials is possible. Other local conditions may be encountered which could require additional removals.

Other Proposed Improvement Areas: For areas such as roadways, at-grade crossings, sidewalks, and other flatwork, we recommend that the upper one foot of the existing soils be scarified and compacted to 95 percent relative compaction (per ASTM D1557). Where the roadbed is within a fill area, these requirements should already be met due to compaction of the newly placed fill (See Section 6.3.3). Where roadbed lies within a cut area, scarification and recompaction may be required to achieve this compaction requirement. If scarification and recompaction are insufficient to achieve required compaction, removal and replacement may be required. The exact extent of removals can best be determined during grading when direct observation, testing, and evaluation of exposed materials are possible. For areas with expansive clayey soils, the subgrade should be prepared in accordance with the recommendations of Section 6.3.5.

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

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

Given the fine-grained soils onsite and high moisture content, this may not be a practical solution.

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

6.3.3 Engineered Fill

All fill soils should be placed in thin (maximum 8-inch loose thickness, except as noted for oversize materials in Section 6.3.4), horizontal lifts with each lift properly moisture conditioned to about two percent above the optimum moisture content and compacted to a minimum of 95 percent relative compaction per ASTM D 1557 (see Section 4.3.2). Subballast and aggregate base should be compacted to a minimum of 95 percent relative compaction.

6.3.4 Fill Material

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

Surficial soils encountered at the boring locations are in general not suitable for use as engineered fill. However, with some regrading, the some onsite soils may be used in the engineering fill provided that they meet the criteria mentioned above.

Structural backfill material should have a Sand equivalent of not less than 20 and should conform to the grading listed in Section 31 20 00 of SCRRA Standard Specifications (2014).

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

6.3.5 Expansive Soils Mitigation

Testing of samples obtained in the vicinity of Simi Valley Station indicated that highly expansive soils should be expected at this location. Pavement and foundations may be susceptible to damage due to the upper expansive soils at the Site.

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

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

6.4 SLOPE GEOMETRY

Based on the topography of the Site along the Project alignment, we do not anticipate major cut or fill slopes in this Projects. For minor slopes (less than 6 feet in height), side slopes should be no steeper than 2H:1V (horizontal to vertical). The specific heights of cut/fill slopes should be evaluated by the geotechnical consultant during final design.

6.5 LATERAL EARTH PRESSURES

Table 6-2 provides a set of equivalent fluid pressure (EFP) values for the preliminary design of earth-retaining structures at the Site. EFP concept is commonly used in the estimation of the lateral earth pressure which a retaining wall or shoring system will be required to resist. EFP is expressed as the unit weight of a fluid (in pcf) which would generate a hydrostatic pressure equal to the anticipated lateral earth pressure at a given depth. This horizontal pressure is applied to a vertical plane extending up from the heel of the wall base, and the weight of soil above the wall heel is included as part of the wall weight. A soil unit weight of 120 pounds per cubic foot (pcf) may be used for calculating the actual weight of the soil over a structure.

EFP values were provided for three wall displacement conditions considering a level backfill. The appropriate condition depends on the type of wall or shoring system selected, and on the installation method. For example, a flexible sheet pile wall system might experience "Active" conditions; a cast-in-place diaphragm wall system might experience "At-Rest" conditions; and the resistance at the toe of the shoring might experience "Passive" conditions. Note that lateral earth pressures will be significantly higher for a sloped backfill condition.

Table 6-2. Lateral Earth Pressures

Condition	Equivalent Fluid Pressure (pcf)	
	Level Backfill	
	Pedestrian Underpass (1)	Other Retaining Walls (2)
Active	60	43
At-Rest	80	64
Passive	250 to a maximum 2,500 psf	250 to a maximum 2,500 psf

Notes:

(1) Assumed native backfill

(2) Assumed granular backfill

(3) Values presented in this table do not include a factor of safety.

(4) Free-draining soil conditions were assumed.

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

It should be noted that the movement required to mobilize passive pressure is approximately 10 times larger than the movement needed to induce earth pressure to the active values. Therefore, a reduction factor of 0.6 is recommended for the passive earth pressure.

For seismic loading, an additional triangular pressure distribution of 20 pcf may be used in addition to the static earth pressures. The seismic pressure should be considered additive to the static equivalent fluid pressures. These seismic earth pressures are applicable for both cantilever and braced conditions. Forces resulting from wall inertia effects are expected to be relatively minor for non-gravity walls and may be ignored in estimating the seismic lateral earth pressure.

Backfills for retaining walls, if any, should be compacted to a minimum of 95 percent relative compaction (per ASTM D1557). Retaining walls should be backfilled with non-expansive granular soils, i.e., backfill Types 1 and 2 per Section 5.2.5, Chapter 8 of AREMA (2019). Backfill using native materials may be difficult due to high existing moisture content, see Section 6.3 for discussion. During construction of retaining walls, the backcut should be made in accordance with the requirements of Cal/OSHA Construction Safety Orders. To mitigate the effects of over-stressing the wall, relatively light construction equipment should be used to achieve the compaction requirement behind retaining walls.

6.6 ENVIROBLOCK RETAINING WALL

A concrete wall consisting of two interlocked Enviroblocks or similar gravity block walls is proposed to the south of the existing rail track between the Simi Valley Station and Ralston Street. Each block is 2.5 ft × 2.5 ft × 5 ft in dimensions with the 5 ft dimension extending along the toe of slope. The Enviroblocks should be stacked vertically with brick pattern overlaps. Compaction and subgrade preparation should be in accordance with the recommendations of this report. Prior to placement of blocks, the upper 8 inches of subgrade soils should be scarified and compacted similar to with normal excavation bottoms in accordance with Section 6.3.

For recommendations regarding the backfill, please see Section 6.5. Other details for the placement of the blocks should be in accordance with the recommendations of the manufacturer. Additional recommendations regarding this retaining wall will be provided in the next design stage. Global and local stability must be checked prior to the final design.

6.7 CONSOLIDATION SETTLEMENTS

Based on the results of the consolidation tests as well as the estimated loads on the proposed pedestrian underpass, the impact of static consolidation is considered low. For lightly loaded structures supported on shallow foundations, anticipated static settlement is estimated to be less than one inch. Differential settlement under static condition is estimated to be less than one-half inch.

6.8 PAVEMENT DESIGN

Installation of the new railroad tracks at each at-grade crossing requires reconstruction of the pavement along the railroad track areas to restore normal traffic flow. Based on the laboratory test results, the R-value of the subsurface soils in the upper five feet (from the existing grade) range between 10 and 73. Considering the onsite soils in the upper five feet and the laboratory test results, we selected two different R-values for the design of pavement sections along the Project alignment. An R-value of 35 is recommended for areas located to the west of Ralston Street. For areas located adjacent to the Simi Valley Station (east of Ralston Street), an R-value of 10 is recommended. Due to the linear nature of the Project, these values should be confirmed by the Project geotechnical engineer during construction.

The hot-mix asphalt (HMA) pavement sections have been designed using the Caltrans Highway Design Manual (2018) with design R-values of 10 and 35, and assumed Traffic Indices (TI) ranging from 5 to 8 for a 20-year design life. TI values should be confirmed by the Project civil engineer or traffic engineering consultant and the existing pavement sections will need to be evaluated during construction to confirm that the recommended pavement thicknesses are not thinner than the existing pavement.

Table 6-3. Generalized Pavement Structural Sections

Traffic Index (TI)	Flexible Pavement Section	
	R-Value = 10 (1)	R-Value = 35 (2)
5.0	3-inch AC over 9-inch AB	3-inch AC over 5-inch AB
6.0	4-inch AC over 11-inch AB	4-inch AC over 6-inch AB
7.0	4-inch AC over 15-inch AB	4-inch AC over 8-inch AB
8.0	5-inch AC over 17-inch AB	5-inch AC over 9-inch AB

Notes:

AC: Asphalt Concrete; AB: Aggregate Base (minimum design R-Value of 78)

(1) R-Value for areas east of Ralston Street

(2) R-Value for areas west of Ralston Street

Subgrade should be prepared in accordance with Section 6.3 of this report. The upper one foot of the existing soils be scarified and compacted to 95 percent relative compaction (per ASTM D1557). Minimum relative compaction requirements for aggregate base should be 95 percent of the maximum laboratory density (ASTM D1557). Soft, clayey subgrade may be exposed and require overexcavation and replacement, or other mitigation as described in Section 6.3. For areas with R-value of 10 or less, subgrade enhancement geotextile is recommended. Subgrade enhancement should be in accordance with Section 213 of the Standard Specifications for Public Works Construction (BNi, 2018). For areas with surficial expansive soils, mitigation measures are presented in Section 6.3.5.

Asphalt concrete and aggregate base should conform to Section 203-6 of the Standard Specifications for Public Works Construction (BNi, 2018). Crushed aggregate base or crushed miscellaneous base should conform to Sections 200-2.2 and 200-2.4 of the Standard Specifications for Public Works Construction (BNi, 2018), respectively.

6.9 BALLAST AND SUB-BALLAST

A stable roadbed is critical to provide the foundation upon which ballast, track, and ties are laid and for support of the track structure with limited deflections. At a minimum the upper 12 inches of roadbed should be properly compacted to at least 95 percent relative compaction (ASTM D1557) prior to placing ballast and sub-ballast. Subgrade should be prepared in accordance with Section 6.3 of this report.

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

The principal purpose of the ballast section is to support the tracks and provide resistance against lateral, longitudinal and vertical movements of ties and rails (i.e., stability). Additionally, the ballast distributes the applied load on a larger surface area resulting in lower pressures applied to the subgrade, provides immediate drainage for the tracks, facilitates maintenance, and provides a necessary degree of elasticity and resilience. Ideal qualities in ballast materials are hardness and toughness, durability or resistance to abrasion and weathering, freedom from deleterious particles (dirt), workability, compactability, cleanability, and availability. Important ballast properties include shape of the ballast particles, degree of sharpness, angularity, and surface texture or roughness. These factors have been shown to have a significant effect on the stability and compactability of aggregates in general. Ballast material properties and placement should conform to SCRRA standard specifications (Section SS 34 11 16) recommended practices. Sub-ballast material properties and placement should conform to SCRRA standard specifications (Section SS 34 11 27) recommended practices.

For preliminary design and based on the SCRRA Engineering Standard Plans (SCRRA, 2016; Drawing E2002) a minimum of 12 inches of ballast (measured from bottom of the concrete ties) over a minimum of 6 inches of sub-ballast may be used for this Project. This recommendation may be updated in the next stages of design when design parameters are provided.

6.10 TRENCH BACKFILL

Utility trenches should be backfilled and compacted with fill material in accordance with Section 10.4, Chapter 8 of AREMA (2019) or Sections 306-12 and 306-13 of the Standard Specifications for Public Works Construction, (“Greenbook”). Additionally, the requirements of SCRRA Excavation Support Guidelines (SCRRA, 2009) applies to all trenches and excavations.

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

Above the bedding zone, trenches can be backfilled with the onsite material, provided it is free of debris, organic material and oversized material greater than 3 inches in largest dimension. Oversized rock (cobbles and/or boulders) should either be removed from the alignment or pulverized for use in backfill. Gravel larger than ¾ inches in diameter should be mixed with at least 80 percent soil by weight passing the No. 4 sieve. We recommend that the materials used for the bedding zone be placed and compacted with mechanical means. Densification by water jetting should not be allowed.

Backfill should be placed in thin lifts, loose lift thickness being compatible with the earthwork equipment but not exceeding 12 inches, moisture-conditioned to up to four (4) percent above optimum moisture content, and mechanically compacted to a minimum 90 percent relative compaction (ASTM D 1557). The upper 12 inches of trench backfill in pavement areas should be compacted to a minimum 95 percent relative compaction.

6.11 Corrosion Measures

A discussion of soil corrosion results is included in Section 4.3.4. The test results included in this report should only be used as a screening process for an indication of soil corrosivity. In general, foundation elements should be designed for a moderately corrosive environment toward buried ferrous metals, and a non-corrosive environment for buried concrete structures. Type II or Type V Portland Cements are appropriate concrete types on the Project, and appropriate strength and mix requirements should be selected based on individual structures' design life and structural requirements. For sensitive buried metallic elements, a corrosion engineer should be consulted.

7.0 CONSTRUCTION CONSIDERATIONS

7.1 GROUNDWATER CONTROL

Based on the current field exploration, groundwater was encountered at approximate depths between 20 and 22 feet below the existing grade in the area near the proposed pedestrian underpass. However, groundwater may exist at shallower depths on a seasonal basis. Relatively shallow groundwater inflow may be controlled by a system of collection ditches and sump pumps. In an event of encountering significant groundwater, the contractor may implement a specific dewatering system.

Dewatering systems should be designed and installed by a specialty contractor. Dewatering within shored excavations may be completed by using shallow well points. Dewatering systems should be properly designed and in conjunction with the shoring system to avoid creating unstable conditions at the bottom of the excavation. Unstable conditions may developed in shored excavations when bottom of excavation is at or near a pervious granular layer that can transmit significant volumes of groundwater. Dewatering systems should be monitored and inspected for effectiveness during the dewatering operation to avoid creation of unstable conditions. In the event of implementing a dewatering system, contractor should consider a monitoring program and instrumentation to monitor groundwater levels within the site, settlements or deformations of the shoring system, and existing adjacent structures. Additionally, the dewatering monitoring program should include routine monitoring for suspended solids and contaminants to ensure compliance with regulatory requirements.

7.2 TEMPORARY EXCAVATIONS AND SHORING

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

In consideration of the type of construction, the most practical method is expected to be excavation bracing. The lateral earth pressure for this type of shoring is estimated as 38H pcf (evenly distributed), where H is the depth of excavation and the resulting lateral pressure distribution is a rectangular pressure. This above lateral pressure is only appropriate for level backfill and a drained condition behind the shoring. The contractor should be responsible for the structural design and safety of all temporary shoring systems. Temporary shoring in the proximity of the railroad track should be designed in accordance with AREMA Chapter 8 Section 28.5 (2019). Shoring should also be designed to resist lateral surcharge from train loading, adjacent vehicular traffic, construction equipment, and existing structures.

7.3 ADDITIONAL GEOTECHNICAL SERVICES

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

- Plans and specifications review;
- Overexcavation and soil removal and/or exposed excavation bottom;

- Pumping or unstable subgrade;
- Placement of compacted fill;
- Footing excavation; and
- When any unusual subsurface conditions are encountered.

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

8.0 LIMITATIONS

This report has been prepared for the use of HDR and the Southern California Regional Rail Authority for the proposed Simi Valley Double Track and Platform Project. This report may not be used by others without the written consent of our client and our firm. The conclusions and recommendations presented in this report are based upon the generally accepted principles and practices of geotechnical engineering utilized by other competent engineers at this time and place. No other warranty is either expressed or implied.

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

9.0 REFERENCES

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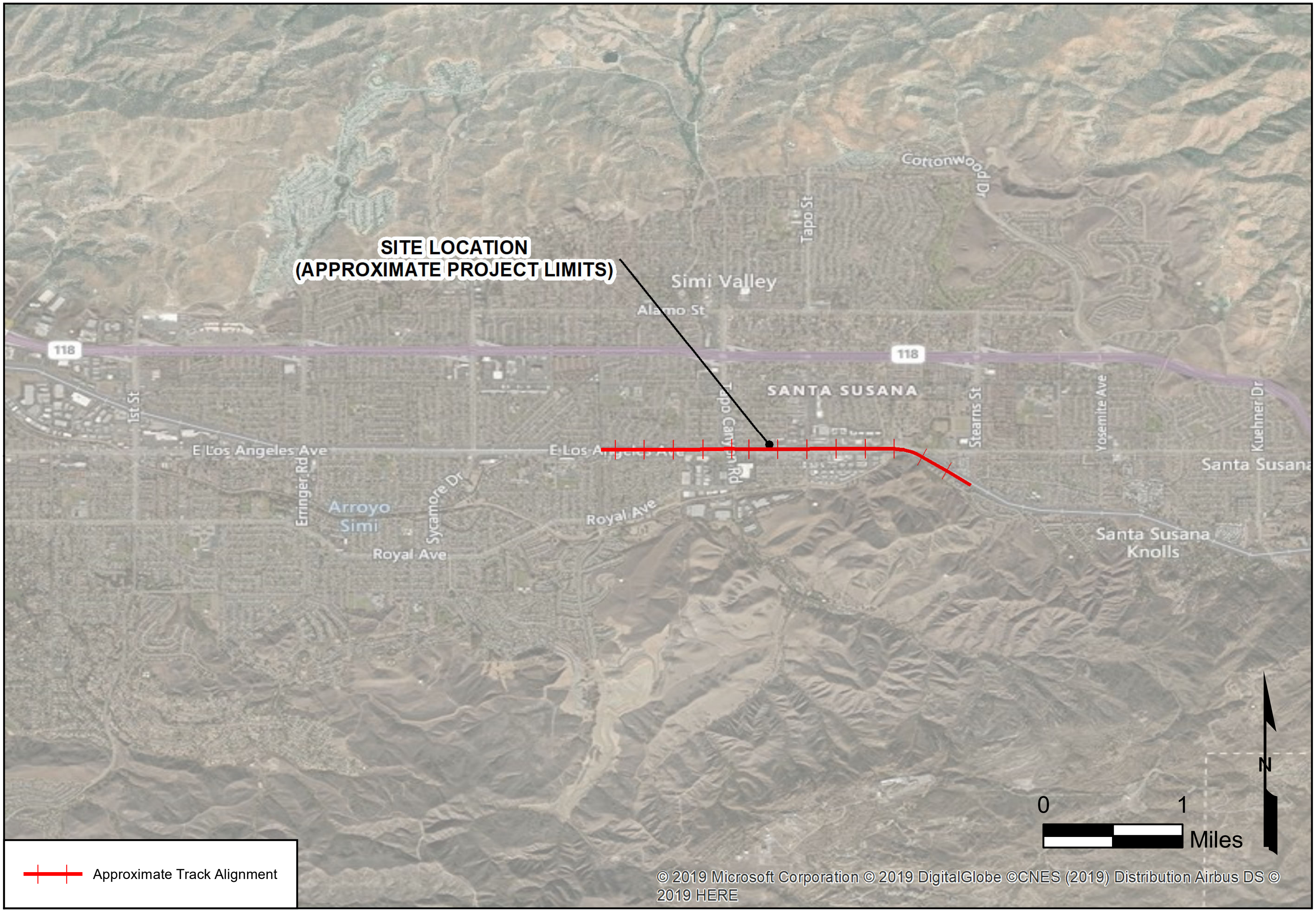
Appendix A

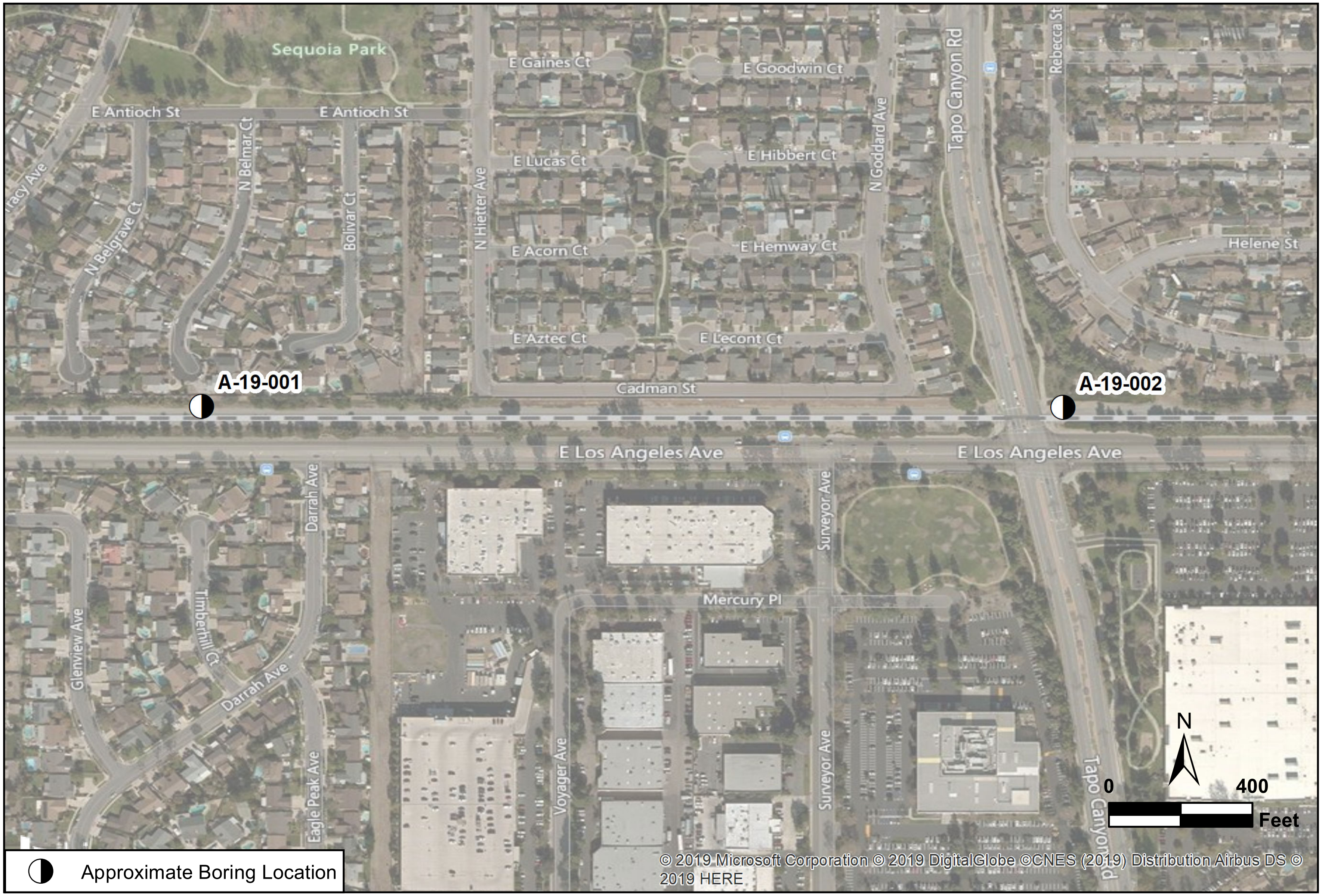
Figures



SITE VICINITY MAP
CTO-48 SCORE VENTURA CORRIDOR
SIMI VALLEY DOUBLE TRACK & PLATFORM
SIMI VALLEY, CALIFORNIA

HDR

Figure 1



BORING LOCATION MAP
CTO-48 SCORE VENTURA CORRIDOR
SIMI VALLEY DOUBLE TRACK & PLATFORM
SIMI VALLEY, CALIFORNIA

Figure 2

BORING LOCATION MAP
CTO-48 SCORE VENTURA CORRIDOR
SIMI VALLEY DOUBLE TRACK & PLATFORM
SIMI VALLEY, CALIFORNIA

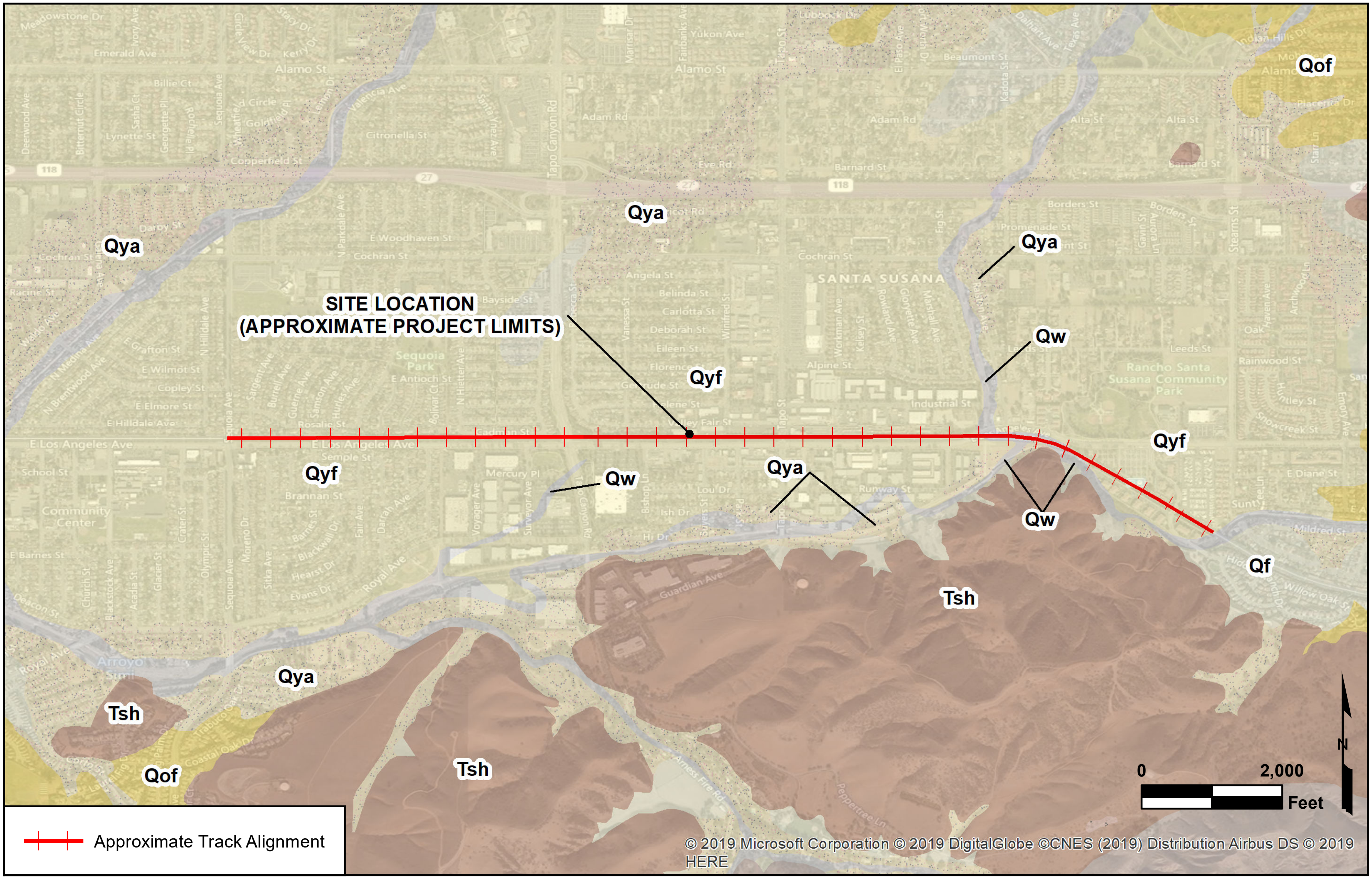
HDR

Figure 3

BORING LOCATION MAP
CTO-48 SCORE VENTURA CORRIDOR
SIMI VALLEY DOUBLE TRACK & PLATFORM
SIMI VALLEY, CALIFORNIA

HDR

Figure 4



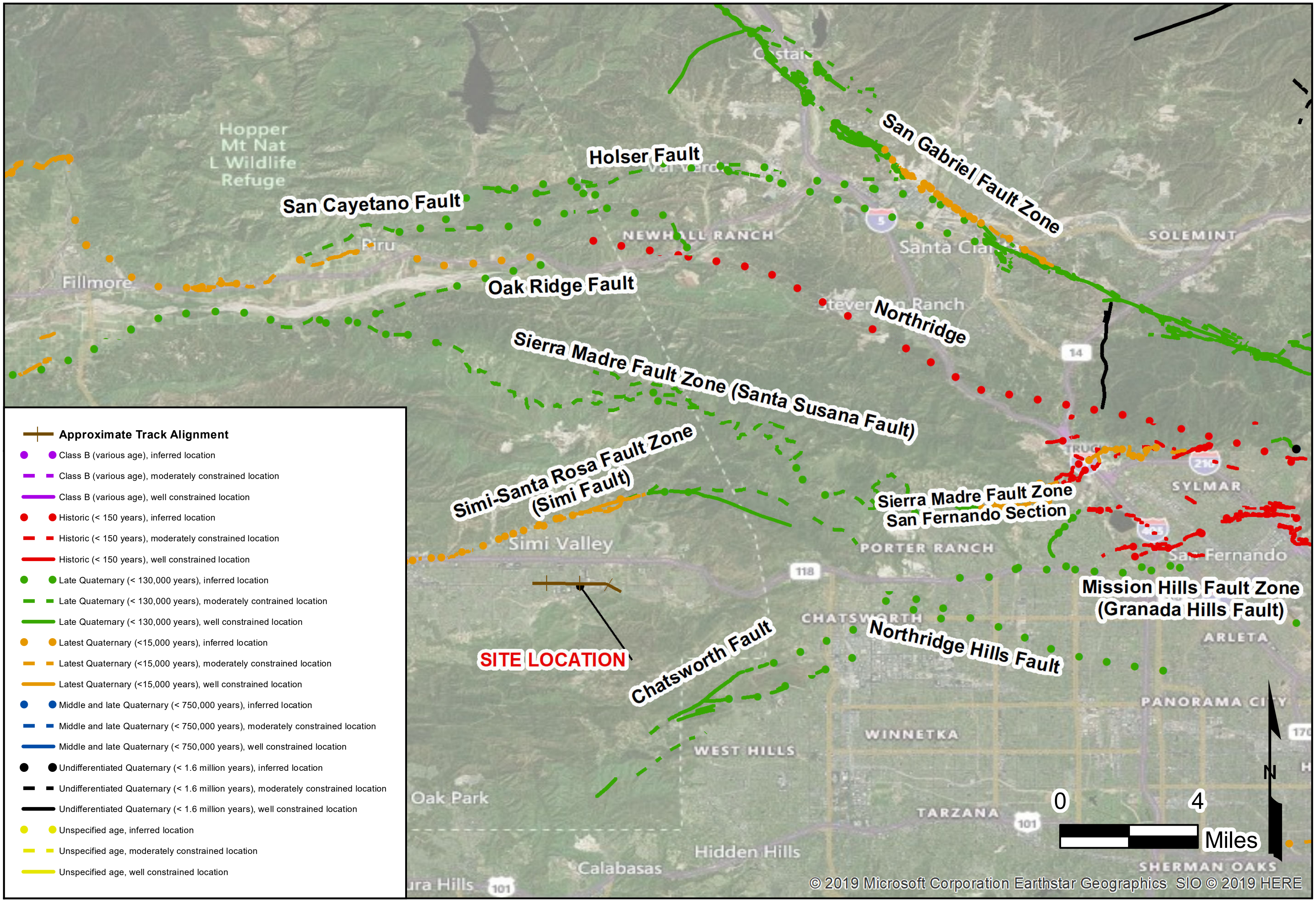
Reference: CGS, 2012

Qyf	**Young Alluvial Fan Deposits** - unconsolidated to slightly consolidated, undissected to slightly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon
Qw	**Alluvial Wash Deposits** - unconsolidated sandy and gravelly sediment deposited in recently active channels of streams and rivers; may contain loose to moderately loose sand and silty sand
Qf	**Alluvial Fan Deposits** - unconsolidated boulders, cobbles, gravel, sand, and silt recently deposited where a river or stream issues from a confined valley or canyon; sediment typically deposited in a fan-shaped cone; gravelly sediment generally more dominant than sandy sediment
Qya	**Young Alluvial Valley Deposits** - unconsolidated to slightly consolidated, undissected to slightly dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers
Tsh	**Fine-grained Tertiary age formations** - includes fine-grained sandstone, siltstone, mudstone, shale, siliceous and calcareous sediments

REGIONAL GEOLOGIC MAP
CTO-48 SCORE VENTURA CORRIDOR
SIMI VALLEY DOUBLE TRACK & PLATFORM
SIMI VALLEY, CALIFORNIA

HDR

Figure 5

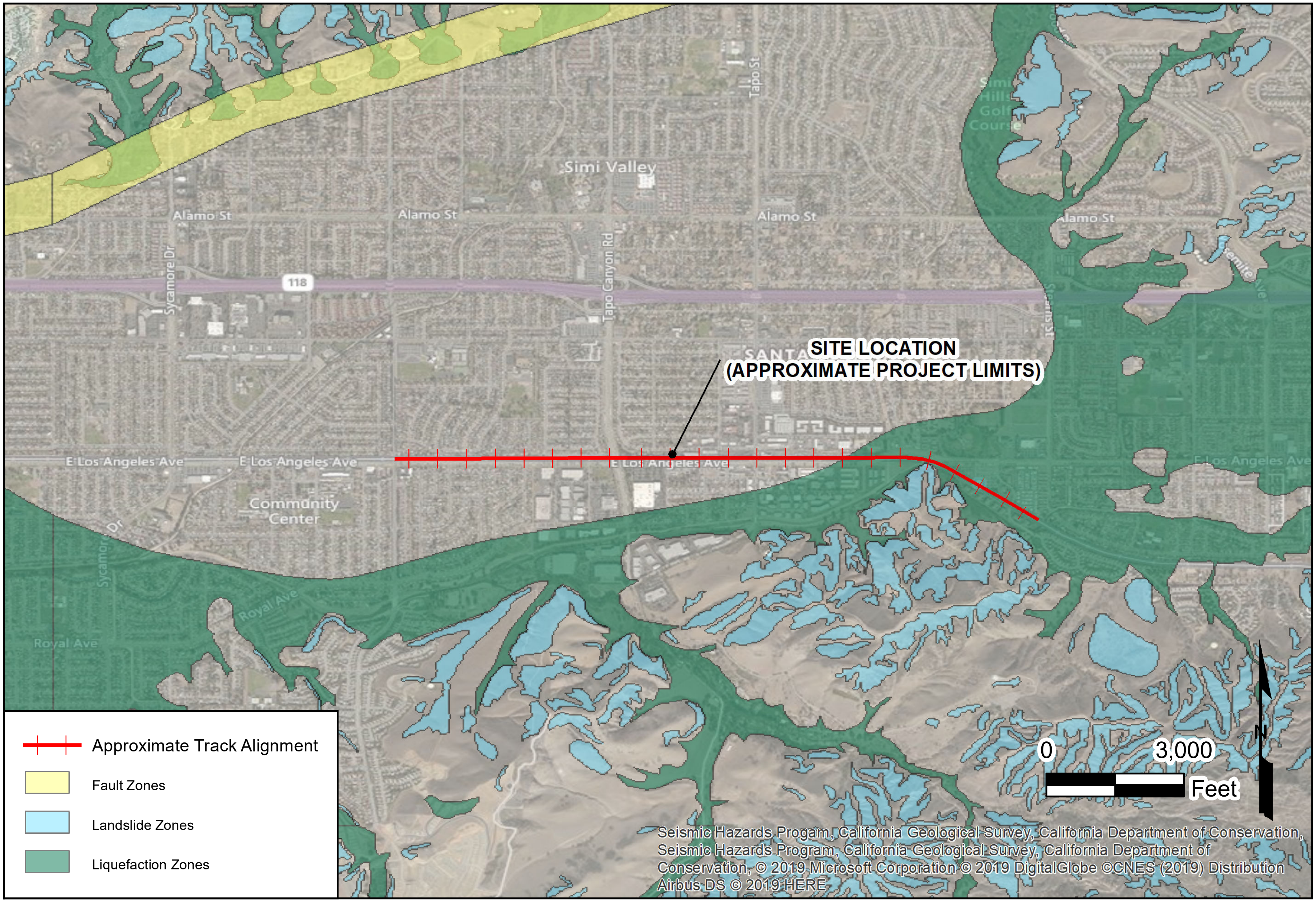


Reference: USGS, 2019b, ArcGIS Database

REGIONAL FAULT MAP
CTO-48 SCORE VENTURA CORRIDOR
SIMI VALLEY DOUBLE TRACK & PLATFORM
SIMI VALLEY, CALIFORNIA

HDR

Figure 6



Reference: CGS, 2019

SEISMIC HAZARD MAP
CTO-48 SCORE VENTURA CORRIDOR
SIMI VALLEY DOUBLE TRACK & PLATFORM
SIMI VALLEY, CALIFORNIA

HDR

Figure 7

Appendix B

Geotechnical Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

MATERIAL TYPES	CRITERIA FOR ASSIGNING SOIL GROUP NAMES			GROUP SYMBOL	SOIL GROUP NAMES & LEGEND
COARSE-GRAINED SOILS >50% RETAINED ON NO. 200 SIEVE	GRAVELS >50% OF COARSE FRACTION RETAINED ON NO 4. SIEVE	CLEAN GRAVELS <5% FINES	$C_u \geq 4$ AND $1 \leq C_c \leq 3$	GW	WELL-GRADED GRAVEL
			$C_u < 4$ AND/OR $1 > C_c > 3$	GP	POORLY-GRADED GRAVEL
		GRAVELS WITH FINES >12% FINES	FINES CLASSIFY AS ML OR MH	GM	SILTY GRAVEL
			FINES CLASSIFY AS CL OR CH	GC	CLAYEY GRAVEL
	SANDS >50% OF COARSE FRACTION PASSES NO 4. SIEVE	CLEAN SANDS <5% FINES	$C_u \geq 6$ AND $1 \leq C_c \leq 3$	SW	WELL-GRADED SAND
			$C_u < 6$ AND/OR $1 > C_c > 3$	SP	POORLY-GRADED SAND
		SANDS AND FINES >12% FINES	FINES CLASSIFY AS ML OR MH	SM	SILTY SAND
			FINES CLASSIFY AS CL OR CH	SC	CLAYEY SAND
FINE-GRAINED SOILS >50% PASSES NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT<50	INORGANIC	PI>7 AND PLOTS>"A" LINE	CL	LEAN CLAY
			PI>4 AND PLOTS<"A" LINE	ML	SILT
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OL	ORGANIC CLAY OR SILT
	SILTS AND CLAYS LIQUID LIMIT>50	INORGANIC	PI PLOTS >"A" LINE	CH	FAT CLAY
			PI PLOTS <"A" LINE	MH	ELASTIC SILT
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OH	ORGANIC CLAY OR SILT
HIGHLY ORGANIC SOILS	PRIMARILY ORGANIC MATTER, DARK IN COLOR, AND ORGANIC ODOR			PT	PEAT

OTHER SYMBOLS

MATERIALS	SAMPLERS
Asphalt	SPT (2" OD)
Aggregate Base	Modified California (3" OD)
Boulders & Cobbles	California (2.5" OD)
Fill	Bulk
Topsoil	Shelby Tube
WELL	HQ Core
Concrete Grout/Fill	Sonic Core
Bentonite/Grout Seal	INITIAL WATER LEVEL MEASUREMENT(WITH DATE)
Sand Pack + Solid Pipe	STABILIZED WATER LEVEL MEASUREMENT(WITH DATE)
Sand Pack + Slotted Pipe	

GRAIN SIZES

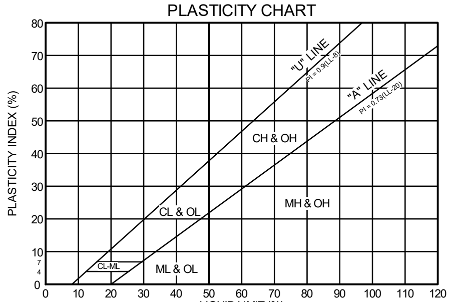
U.S. STANDARD SIEVE	200	40	10	4	3/4"	3"	12"
SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

PENETRATION RESISTANCE

SAND & GRAVEL		SILT & CLAY		
RELATIVE DENSITY	BLOWS/FOOT (N_{60})	CONSISTENCY	BLOWS/FOOT*	UNC. COMP. STRENGTH (TSF)
VERY LOOSE	0 - 4	VERY SOFT	0 - 1	0 - 1/4
LOOSE	5 - 10	SOFT	2 - 4	1/4 - 1/2
MEDIUM DENSE	11 - 30	MEDIUM STIFF	5 - 8	1/2 - 1
DENSE	31 - 50	STIFF	9 - 15	1 - 2
VERY DENSE	OVER 50	VERY STIFF	16 - 30	2 - 4
		HARD	OVER 30	OVER 4

* NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 INCHES TO DRIVE A 2 INCH O.D. (1-3/8 INCH I.D.) SPLIT-BARREL SAMPLER THE LAST 12 INCHES OF AN 18-INCH DRIVE (ASTM-1586 STANDARD PENETRATION TEST).

PLASTICITY CHART



NOTES

bgs	BELOW GROUND SURFACE
c	COHESION
CD	CONSOLIDATED DRAINED TRIAXIAL
CN	CONSOLIDATION
CR	CORROSIVITY
CU	CONSOLIDATED UNDRAINED TRIAXIAL
DS	DIRECT SHEAR
EI	EXPANSION INDEX
HY	HYDROMETER
MD	MAX DENSITY (COMPACTION)
N_{60}	BLOW COUNT, Corrected for Hammer Energy Only
PI	PLASTICITY INDEX
PR	PERMEABILITY
RV	R-VALUE
SA	SIEVE ANALYSIS
SE	SAND EQUIVALENT
TC	CYCLIC TRIAXIAL
TR	TIME RATE OF CONSOLIDATION
UC	UNCONFINED COMPRESSION
UU	UNCONSOLIDATED UNDRAINED TRIAXIAL

INCREASING VISUAL MOISTURE CONTENT

WET
MOIST
DRY

COMPONENT PERCENTAGE

MOSTLY	>50%
SOME	30 - 50%
LITTLE	15 - 29%
FEW	5 - 14%
TRACE	<5%

Boring Legend

CTO 48 SCORE Ventura Corridor Segment 1- Simi Valley Station
Simi Valley, CA

Date
DEC 2019

Figure

LOGGED BY: MF | DATE: START 11/20/19 END 11/20/19

STATION & OFFSET: NA, NA | LATITUDE: 34.27204 | LONGITUDE: -118.72361 | ELEVATION (ft): 950

DRILL RIG: CME-75 | DRILL METHOD: HSA | DRILLING COMPANY: 2R Drilling | BOREHOLE DEPTH (ft): 21.5

CASING TIP DEPTH: NA | BIT DIAMETER: 8" | GROUNDWATER DATA: DEPTH: DEPTH:

HAMMER TYPE: Automatic | HAMMER EFFICIENCY: 91% | NOT ENCOUNTERED [X] | TIME: TIME:

CHECKED BY (DATE): MK | EFFICIENCY MEASURED [X] | GW NOT MEASURED [ ] | DATE: DATE:

ELEVATION (ft)	DEPTH (ft)	SAMPLER	SAMPLE NO./ CORE RUN	FIELD BLOWS/6 in	POCKET PEN (ksf)	% FINES	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	OTHER TESTS	MATERIAL GRAPHIC	DESCRIPTION	REMARKS
	0										Sandy SILT (ML); brown; moist; coarse to fine SAND; low plasticity; trace fine GRAVEL	
			B			59			SA			
			1	4 5 6			80	13.7	MD RV CR		stiff	
945	5		2	5 7 7		27	99	9.7	SA		Silty SAND (SM); medium dense; light brown; coarse to fine SAND; trace fine GRAVEL	
940	10		3	2 2 3				14.5			loose; trace CLAY	
935	15		4	3 6 7							Clayey SAND (SC); medium dense; brown; moist; fine SAND	
930	20		5	3 2 3							Lean CLAY (CL); medium stiff; black; moist; low plasticity; trace fine SAND	

Boring terminated at 21.5 feet bgs.
Groundwater not encountered.
Boring backfilled with soil cuttings.

LOGGED BY: MF				DATE: START 11/20/19	END 11/20/19		
STATION & OFFSET: NA, NA		LATITUDE: 34.27203		LONGITUDE: -118.71703		ELEVATION (ft): 958	
DRILL RIG: CME-75		DRILL METHOD: HSA		DRILLING COMPANY: 2R Drilling		BOREHOLE DEPTH (ft): 14.5	
CASING TIP DEPTH: NA		BIT DIAMETER: 8"		GROUNDWATER DATA:		DEPTH:	DEPTH:
HAMMER TYPE: Automatic		HAMMER EFFICIENCY: 91%		NOT ENCOUNTERED ☒		TIME:	TIME:
CHECKED BY (DATE): MK		EFFICIENCY MEASURED ☒		GW NOT MEASURED ☐		DATE:	DATE:

ELEVATION (ft)	DEPTH (ft)	SAMPLER	SAMPLE NO./ CORE RUN	FIELD BLOWS/6 in	POCKET PEN (ksf)	% FINES	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	OTHER TESTS	MATERIAL GRAPHIC	DESCRIPTION	REMARKS
	0										Sandy SILT (ML); brown; moist; coarse to fine SAND; trace CLAY; trace fine GRAVEL (FILL)	Boring relocated 5 feet to the north due to abandoned utility cables. Hand auger from 0 to 5 feet
955												
	5		1	5 5 7		64		14.3	SA		stiff	
950												
											Clayey SAND (SC); loose; dark brown; moist; fine SAND; few CLAY	
	10			4 4 5								No recovery
945				14 8 16							medium dense	No recovery

Boring terminated at 14.5 feet bgs due to unknown obstruction.
Groundwater not encountered.
Boring backfilled with soil cuttings.

HDR BORING LOG IRVINE 2019-18 CTO 48 SCORE VENTURA CORRIDOR.GPJ FOLSOM 3-30-11.GDT 1/21/20

HDR

Boring Log

CTO 48 SCORE Ventura Corridor Segment 1- Simi Valley Station
Simi Valley, CA

Date
DEC 2019

Boring
A-19-002

LOGGED BY: MF | DATE: START 11/20/19 END 11/20/19
STATION & OFFSET: NA, NA | LATITUDE: 34.27200 | LONGITUDE: -118.70925 | ELEVATION (ft): 968
DRILL RIG: CME-75 | DRILL METHOD: HSA | DRILLING COMPANY: 2R Drilling | BOREHOLE DEPTH (ft): 21.5
CASING TIP DEPTH: NA | BIT DIAMETER: 8" | GROUNDWATER DATA: DEPTH: DEPTH:
HAMMER TYPE: Automatic | HAMMER EFFICIENCY: 91% | NOT ENCOUNTERED [X] | TIME: TIME:
CHECKED BY (DATE): MK | EFFICIENCY MEASURED [X] | GW NOT MEASURED [ ] | DATE: DATE:

ELEVATION (ft)	DEPTH (ft)	SAMPLER	SAMPLE NO./ CORE RUN	FIELD BLOWS/6 in	POCKET PEN (ksf)	% FINES	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	OTHER TESTS	MATERIAL GRAPHIC	DESCRIPTION	REMARKS
	0										Silty SAND with GRAVEL (SM); brown; moist; fine SAND; little coarse to fine GRAVEL	Hand auger from 0 to 5 feet.
965			B			34		6.4	SA MD EI RV SE			
	5		1	3 2 5			99	8.5	DS		Silty SAND (SM); loose; brown; moist; medium to fine SAND; trace root	
960												
	10		2	1 1 2		47		11.9	PI		coarse to fine SAND; trace fine GRAVEL	
955												
	15		3	3 3 6								
950											Poorly-graded SAND with SILT (SP-SM); olive brown; moist; coarse to fine SAND; few fine GRAVEL	
	20		4	4 10 17							dense	

Boring terminated at 21.5 feet bgs.
Groundwater not encountered.
Boring backfilled with soil cuttings.

Boring Log

CTO 48 SCORE Ventura Corridor Segment 1- Simi Valley Station
Simi Valley, CA

Date
DEC 2019

Boring
A-19-003

LOGGED BY: MF | DATE: START 11/20/19 END 11/20/19
STATION & OFFSET: NA, NA | LATITUDE: 34.27206 | LONGITUDE: -118.70185 | ELEVATION (ft): 981
DRILL RIG: CME-75 | DRILL METHOD: HSA | DRILLING COMPANY: 2R Drilling | BOREHOLE DEPTH (ft): 21.5
CASING TIP DEPTH: NA | BIT DIAMETER: 8" | GROUNDWATER DATA: | DEPTH: | DEPTH:
HAMMER TYPE: Automatic | HAMMER EFFICIENCY: 91% | NOT ENCOUNTERED [X] | TIME: | TIME:
CHECKED BY (DATE): MK | EFFICIENCY MEASURED [X] | GW NOT MEASURED [ ] | DATE: | DATE:

ELEVATION (ft)	DEPTH (ft)	SAMPLER	SAMPLE NO./ CORE RUN	FIELD BLOWS/6 in	POCKET PEN (ksf)	% FINES	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	OTHER TESTS	MATERIAL GRAPHIC	DESCRIPTION	REMARKS
980	0										Silty SAND (SM); brown; moist; coarse to fine SAND; trace fine GRAVEL	Hand auger from 0 to 5 feet
975	5		1	4 5 5		39		4.2	SA		medium dense	
970	10		2	8 12 13			103	7.8			fine SAND	
965	15		3	3 5 6		46						
960	20		4	9 14 17								

Boring terminated at 21.5 feet bgs.
Groundwater not encountered.
Boring backfilled with soil cuttings.

HDR

Boring Log

CTO 48 SCORE Ventura Corridor Segment 1- Simi Valley Station
Simi Valley, CA

Date
DEC 2019

Boring
A-19-004

LOGGED BY: MF   DATE: START 11/20/19 END 11/20/19

STATION & OFFSET: NA, NA   LATITUDE: 34.26971 LONGITUDE: -118.69441 ELEVATION (ft): 986

DRILL RIG: CME-75   DRILL METHOD: HSA   DRILLING COMPANY: 2R Drilling   BOREHOLE DEPTH (ft): 50

CASING TIP DEPTH: NA   BIT DIAMETER: 8"   GROUNDWATER DATA: DEPTH: 21.8 ft   DEPTH:

HAMMER TYPE: Automatic   HAMMER EFFICIENCY: 91%   NOT ENCOUNTERED ☐   TIME: TIME:

CHECKED BY (DATE): MK   EFFICIENCY MEASURED ☒   GW NOT MEASURED ☐   DATE: DATE:

ELEVATION (ft)	DEPTH (ft)	SAMPLER	SAMPLE NO./ CORE RUN	FIELD BLOWS/6 in	POCKET PEN (ksf)	% FINES	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	OTHER TESTS	MATERIAL GRAPHIC	DESCRIPTION	REMARKS
985	0										Sandy lean CLAY (CL); brown; moist; fine SAND; low plasticity; trace GRAVEL (FILL)	
			B			61			SA			
			1	3 4 5			97	21.1	MD EI RV			
											Sandy lean CLAY (CL); medium stiff; brown; moist; fine SAND; low plasticity (NATIVE)	
980	5		2	2 3 5		67	102	16.1	PI SA CR DS			
											Lean CLAY (CL); medium stiff; brown; moist; low plasticity; trace fine SAND	
975	10		3	1 1 3		98		33.3				
											Lean CLAY with SAND (CL); medium stiff; brown; moist; fine SAND; low plasticity	
970	15		4	2 2 5	2.0	77	85	32.1	PI CR CN			
965	20		5	1 1 1		82		27.6	PI			
								▽			soft; wet	
											Clayey SAND (SC); medium dense; olive brown; wet; medium to fine SAND	
960	25		6	4 6 7			107	20.6	DS			
											Fat CLAY (CH); soft; brown; wet; high plasticity;	
	30											

ELEVATION (ft)	DEPTH (ft)	SAMPLER	SAMPLE NO./ CORE RUN	FIELD BLOWS/6 in	POCKET PEN (ksf)	% FINES	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	OTHER TESTS	MATERIAL GRAPHIC	DESCRIPTION	REMARKS
955	30		7	1 1 2		90		36.2	PI		few fine SAND	
											Lean CLAY (CL); stiff; brown; wet; low plasticity; trace fine SAND	
950	35		8	3 5 6		98	81	41.0	CN			
											Well-graded SAND with SILT (SW-SM); medium dense; olive gray; wet; coarse to fine SAND; few coarse to fine GRAVEL	
945	40		9	2 5 5		12			SA			
940	45		10	6 12 17			112	19.7				
			11	5 6 12							medium to fine SAND	
	50											

Boring terminated at 50 feet bgs
Groundwater measured at 21.8 feet bgs.
Boring backfilled with cement-bentonite grout.

Boring Log

CTO 48 SCORE Ventura Corridor Segment 1- Simi Valley Station
Simi Valley, CA

Date
DEC 2019

Boring
A-19-005

LOGGED BY: MF				DATE: START 11/20/19	END 11/20/19
STATION & OFFSET: NA, NA		LATITUDE: 34.26937	LONGITUDE: -118.69381	ELEVATION (ft): 988	
DRILL RIG: CME-75	DRILL METHOD: HSA	DRILLING COMPANY: 2R Drilling		BOREHOLE DEPTH (ft): 21.5	
CASING TIP DEPTH: NA	BIT DIAMETER: 8"	GROUNDWATER DATA:		DEPTH: 20.0 ft	DEPTH:
HAMMER TYPE: Automatic	HAMMER EFFICIENCY: 91%	NOT ENCOUNTERED ☐		TIME:	TIME:
CHECKED BY (DATE): MK	EFFICIENCY MEASURED ☒	GW NOT MEASURED ☐		DATE:	DATE:

ELEVATION (ft)	DEPTH (ft)	SAMPLER	SAMPLE NO./ CORE RUN	FIELD BLOWS/6 in	POCKET PEN (ksf)	% FINES	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	OTHER TESTS	MATERIAL GRAPHIC	DESCRIPTION	REMARKS
988	0										Sandy lean CLAY (CL); dark gray; moist; coarse to fine SAND; low plasticity; trace coarse to fine GRAVEL (FILL)	
											trace plastic pieces	
985			1	4 9 7		58	111	14.3	SA		very stiff	
	5		2	4 5 5	4.0	75	96	21.2	PI CN		Lean CLAY with SAND (CL); stiff; brown; moist; fine SAND; low plasticity (NATIVE)	
980												
	10		3	2 3 4				35.5				
975											Lean CLAY (CL); medium stiff; brown; moist; low plasticity; trace fine SAND	
	15		4	4 6 7	2.0	99	86	33.1	DS			
970												
	20		5	1 2 3				▽ 39.0			medium stiff; wet	

Boring terminated at 21.5 feet bgs.
Groundwater measured at 20 feet bgs.
Boring backfilled with cement-bentonite grout.

HDR

Boring Log

CTO 48 SCORE Ventura Corridor Segment 1- Simi Valley Station
Simi Valley, CA

Date
DEC 2019

Boring
A-19-006

Appendix C

Geotechnical Laboratory Testing Results

TABLE C-1
SUMMARY OF GEOTECHNICAL LABORATORY DATA

Project: CTO-48 (Simi Valley)
Project No.: 10193167

Boring No.	Sample Depth (ft)	Soil Type (USCS)	Sample Elev. (ft)	Moisture Content (%)	Dry Density (pcf)	Gradation			Compaction		Atterberg Limits			Direct Shear Strength Peak		Direct Shear Strength Ultimate		Consolidation		R-value	Sand Equivalent	Expansion Index	Corrosion Analyses			
						Gravel (%)	Sand (%)	Fines (%)	Max. Dry Density (pcf)	Optimum Moisture Content (%)	LL	PL	PI	ϕ' (deg)	c' (psf)	ϕ' (deg)	c' (psf)	Swell (+) or Collapse (-) (%)	Swell or Collapse Pressure (ksf)				pH	Min Resistivity (Ω-cm)	Sulfate (ppm)	Chloride (ppm)
A-19-001	0-5	ML	950	13.7	80.0	1	40	59	114.6	14.9										38			7.4	1680	47	0.7
A-19-001	5.0	SM	945	9.7	99.0	4	69	27																		
A-19-001	10.0		940	14.5																						
A-19-001	15.0		935																							
A-19-001	20.0		930																							
A-19-002	0-5		958																							
A-19-002	5.0	ML	953	14.3		4	32	64																		
A-19-002	10.0		948																							
A-19-002	15.0		943																							
A-19-002	20.0		938																							
A-19-003	0 - 5	SM	968	6.4		22	44	34	129.5	8.7										73	14	2				
A-19-003	5.0	SM	963	8.5	99.3									32	100	32	100									
A-19-003	10.0		958	11.9				47			NP	NP	NP													
A-19-003	15.0		953																							
A-19-003	20.0		948																							
A-19-004	5.0	SM	976	4.2		1	60	39																		
A-19-004	10.0		971	7.8	103.3																					
A-19-004	15.0		966					46																		
A-19-004	20.0		961																							
A-19-005	0.5-5	CL	986	21.1	97.0	3	36	61	116.4	14.1										10		109	7.4	1560	46	3.1
A-19-005	5.0	CL	981	16.1	101.9	0	33	67			27	17	10	27	200	27	200									
A-19-005	10.0		976	33.0				98																		
A-19-005	15.0	CL	971	32.1	85.1			77			35	18	17					0.1	2							
A-19-005	20.0	CL	966	27.6				82			40	19	21										7.6	1440	95	3.6
A-19-005	25.0	SC	961	20.6	107.4									29	750	29	150									
A-19-005	30.0	CH	956	36.2				90			53	22	31													
A-19-005	35.0	CL	951	41.0	80.9			98										0.35	2							
A-19-005	40.0	SW-SM	946			12	76	12																		
A-19-005	45.0	SW-SM	941	19.7	111.5																					
A-19-006	3.0	CL	985	14.3	111.1	4	38	58																		
A-19-006	5.0	CL	983	21.2	95.8			75			36	20	16					0.16	0.5							
A-19-006	10.0		978	35.5																						
A-19-006	15.0	CL	973	33.1	86.4			99						20	250	20	250									
A-19-006	20.0		968	39.0																						

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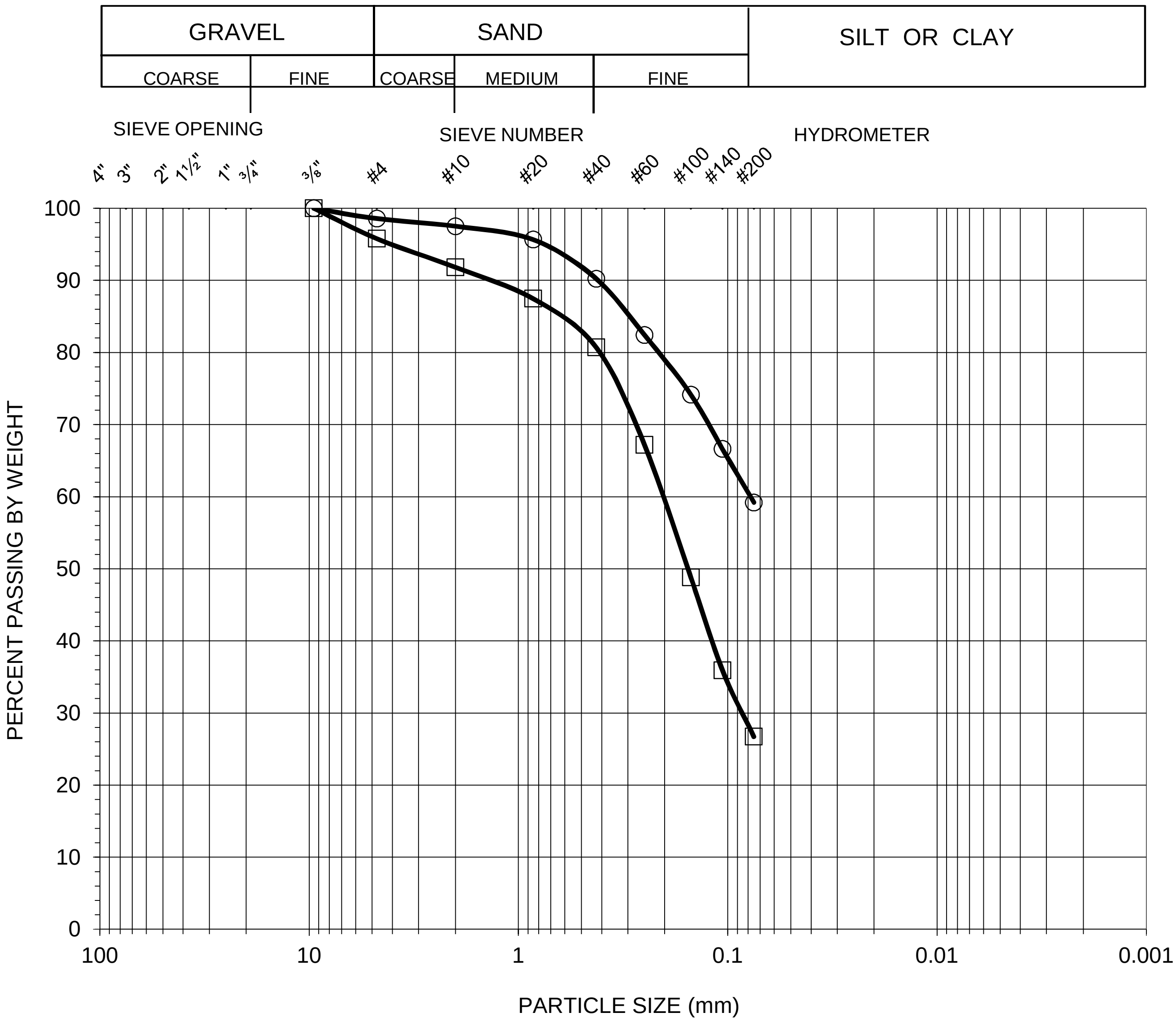
GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name:	HDR	Tested by:	NR	Date:	12/05/19
Project Name:	CTO-48 SCORE Ventura Corridor	Computed by:	NR	Date:	12/09/19
Project Number:	10193167	Checked by:	AP	Date:	12/11/19



Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Type U.S.C.S
				Gravel	Sand	Silt & Clay		
○	A-19-001	B	0-5	1	40	59	N/A	ML
□	A-19-001	2	5	4	69	27	N/A	SM

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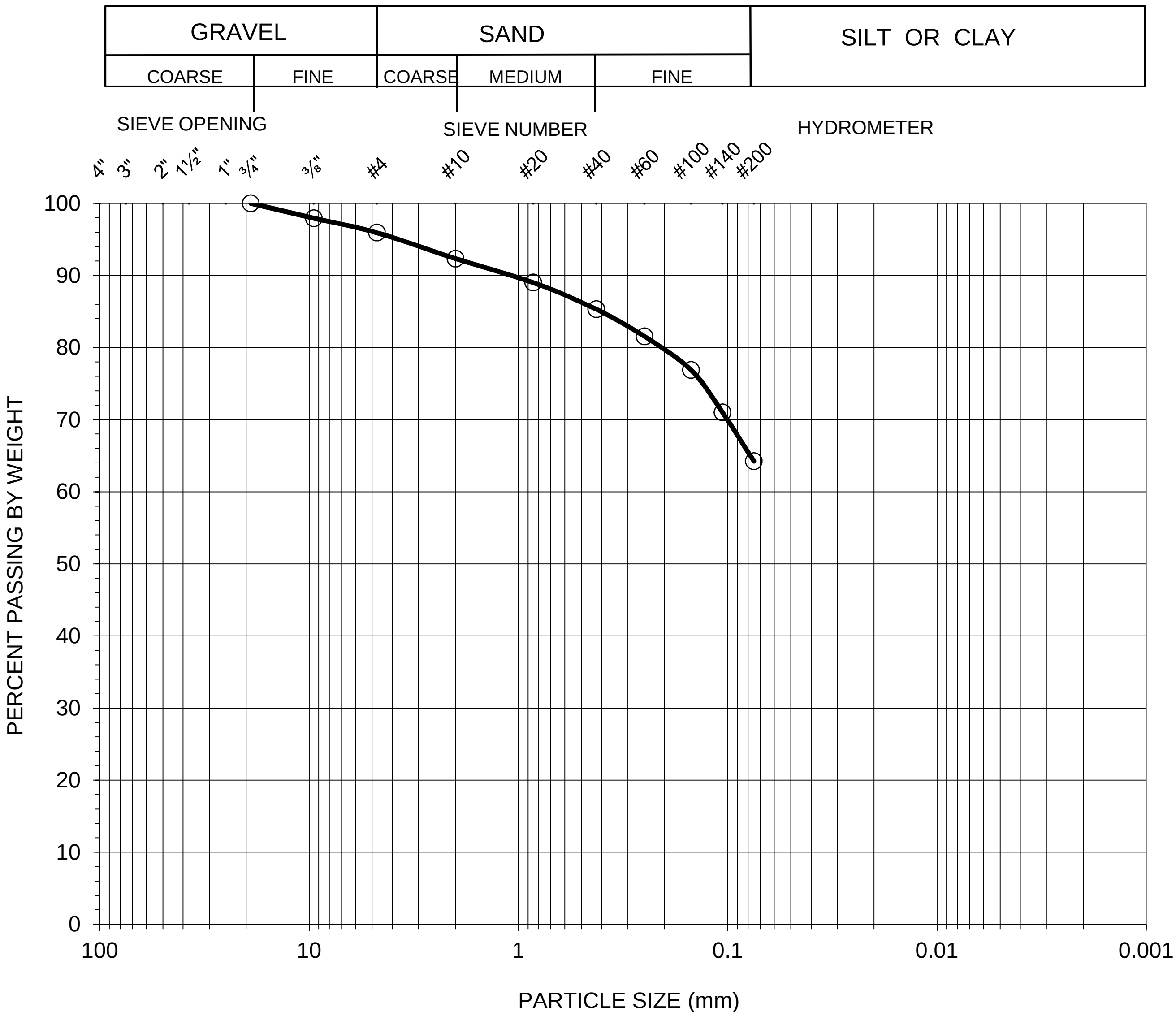
GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name:	HDR	Tested by:	NR	Date:	12/05/19
Project Name:	CTO-48 SCORE Ventura Corridor	Computed by:	NR	Date:	12/09/19
Project Number:	10193167	Checked by:	AP	Date:	12/11/19



Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Type U.S.C.S
				Gravel	Sand	Silt & Clay		
○	A-19-002	1	5	4	32	64	N/A	ML

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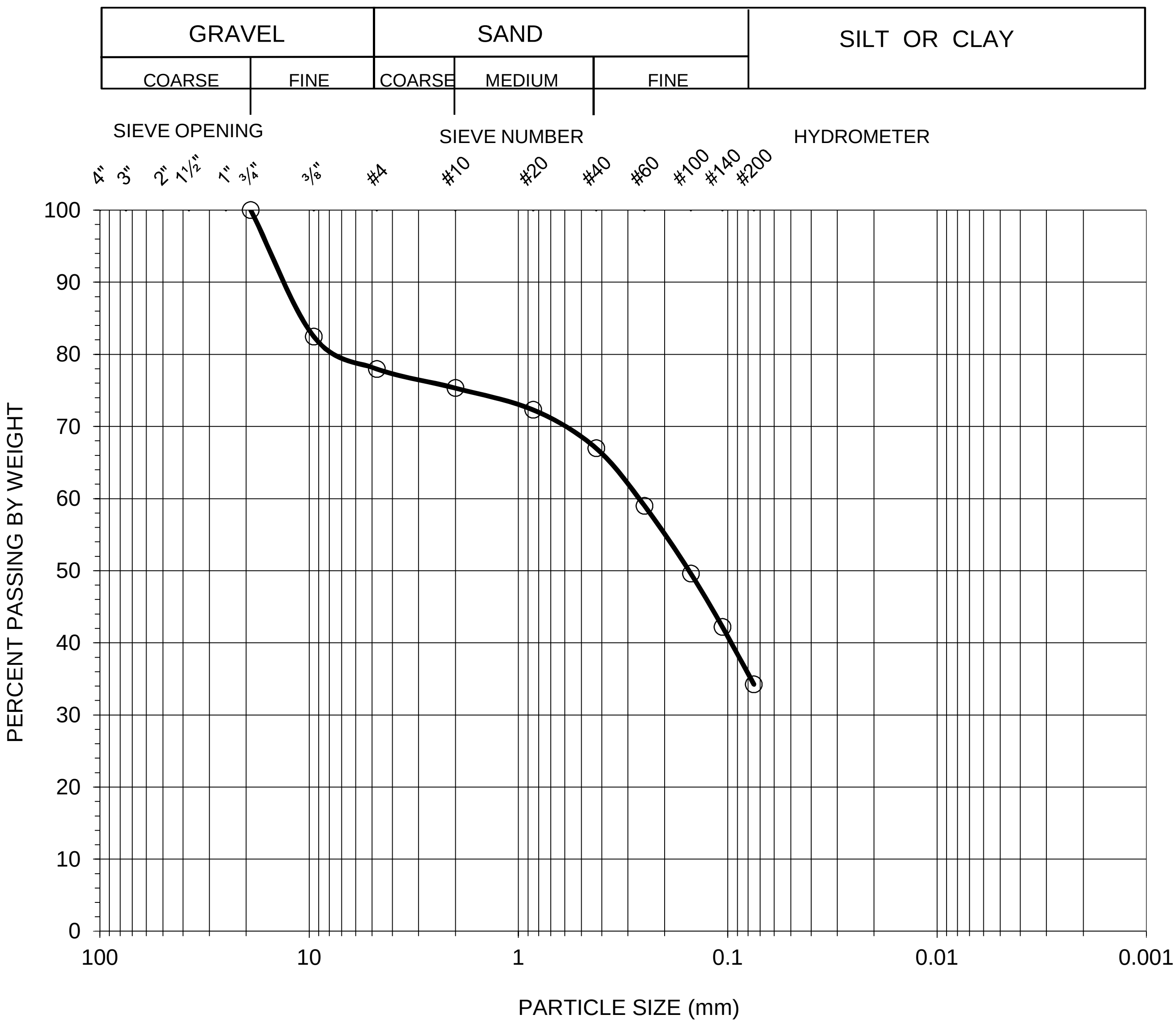
GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name:	HDR	Tested by:	NR	Date:	12/05/19
Project Name:	CTO-48 Score Ventura Corridor	Computed by:	NR	Date:	12/09/19
Project Number:	10193167	Checked by:	AP	Date:	12/11/19



Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Type U.S.C.S
				Gravel	Sand	Silt & Clay		
○	A-19-003	B	0-5	22	44	34	N/A	SM

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GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name:	HDR	Tested by:	NR	Date:	12/05/19
Project Name:	CTO-48 Score Ventura Corridor	Computed by:	NR	Date:	12/09/19
Project Number:	10193167	Checked by:	AP	Date:	12/11/19

Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Type U.S.C.S
				Gravel	Sand	Silt & Clay		
○	A-19-004	1	5	1	60	39	N/A	SM

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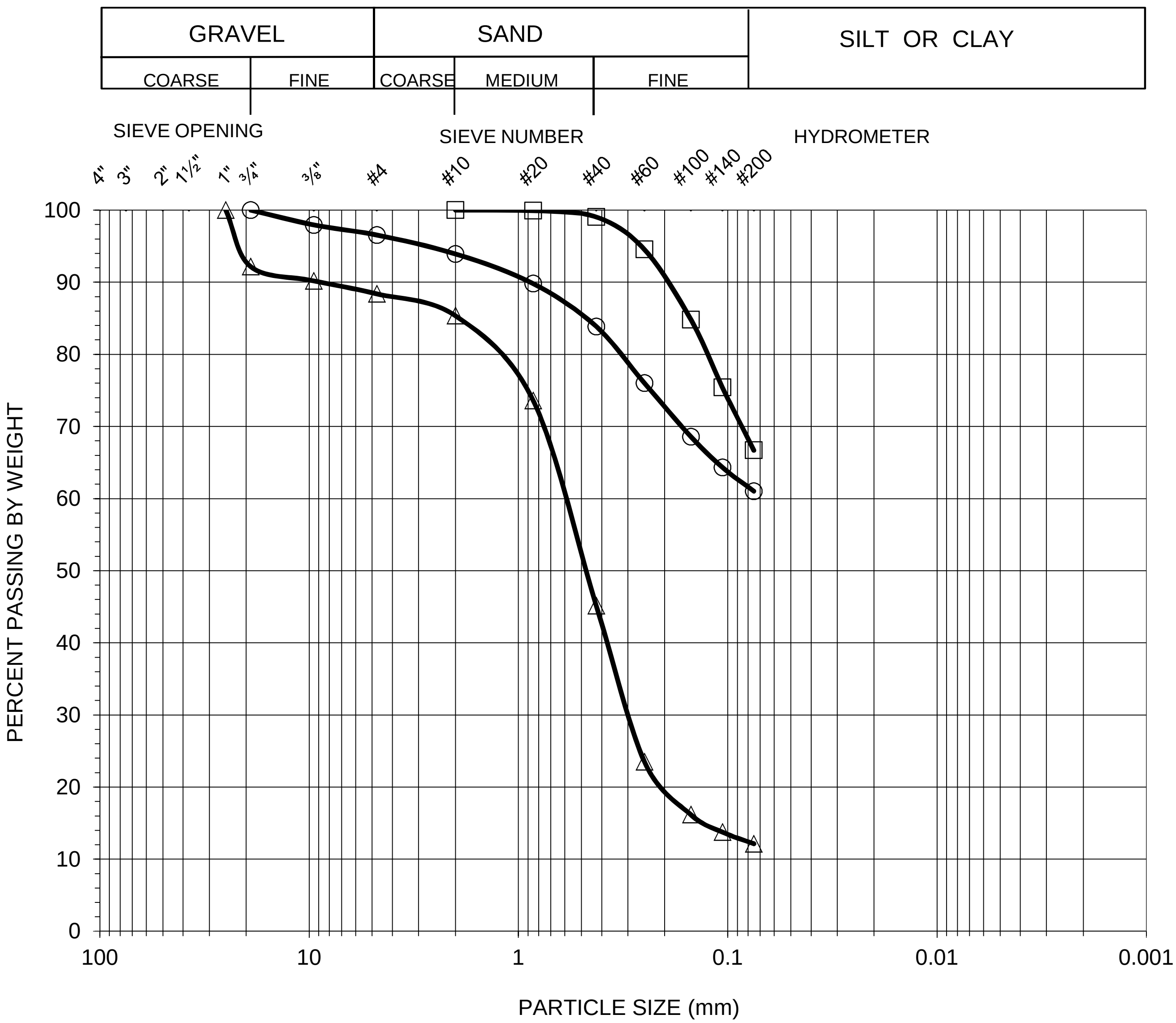
GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name:	HDR	Tested by:	NR	Date:	12/05/19
Project Name:	CTO-48 SCORE Ventura Corridor	Computed by:	NR	Date:	12/09/19
Project Number:	10193167	Checked by:	AP	Date:	12/11/19



Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Type U.S.C.S
				Gravel	Sand	Silt & Clay		
○	A-19-005	B	0-5	3	36	61	N/A	CL*
□	A-19-005	2	5	0	33	67	27:17:10	CL
△	A-19-005	9	40	12	76	12	N/A	SW-SM

**Note: Based on visual classification of sample*

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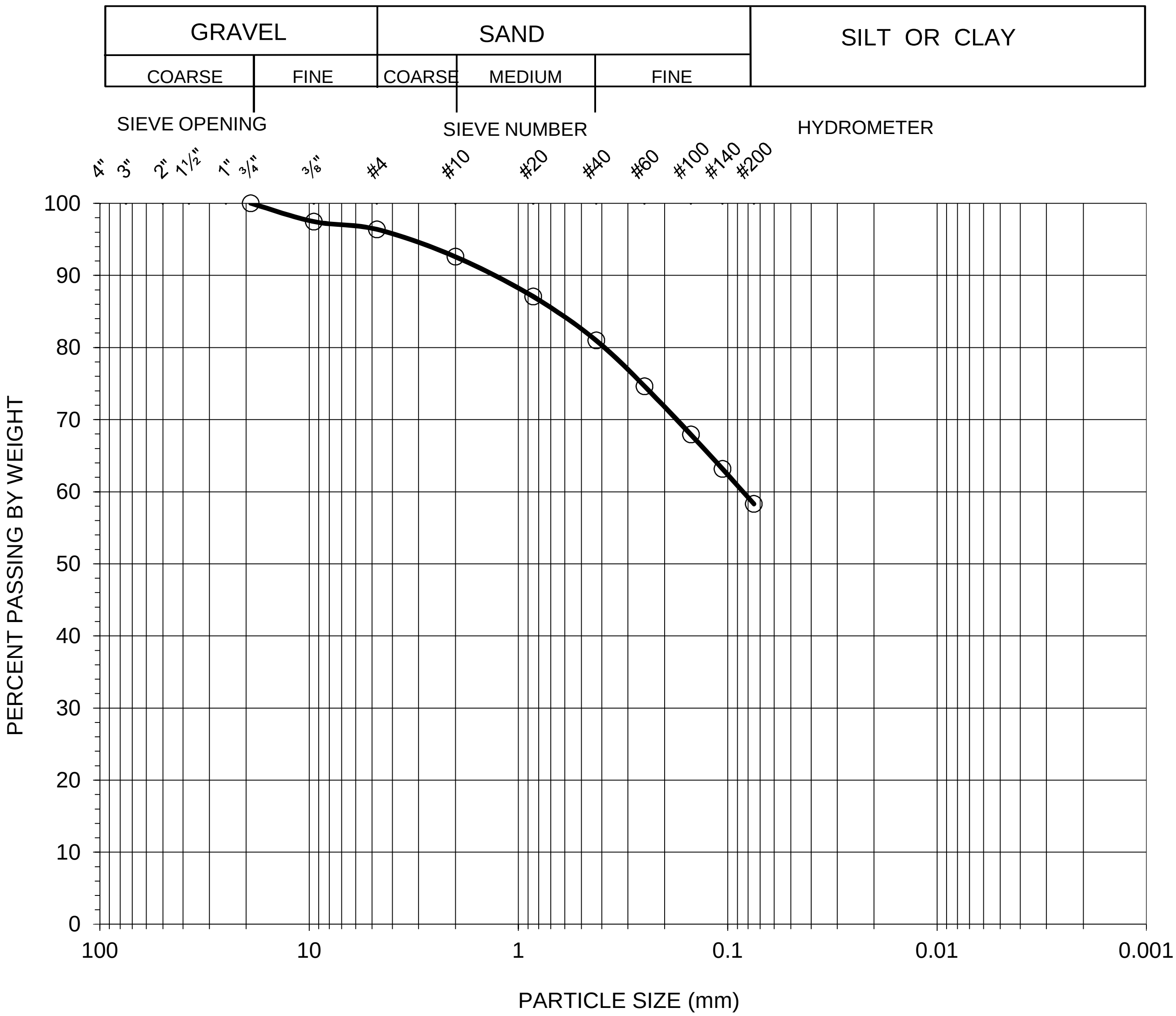
GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name:	HDR	Tested by:	NR	Date:	12/05/19
Project Name:	CTO-48 SCORE Ventura Corridor	Computed by:	NR	Date:	12/09/19
Project Number:	10193167	Checked by:	AP	Date:	12/11/19



Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Type U.S.C.S
				Gravel	Sand	Silt & Clay		
○	A-19-006	1	3	4	38	58	N/A	CL*

**Note: Based on visual classification of sample*

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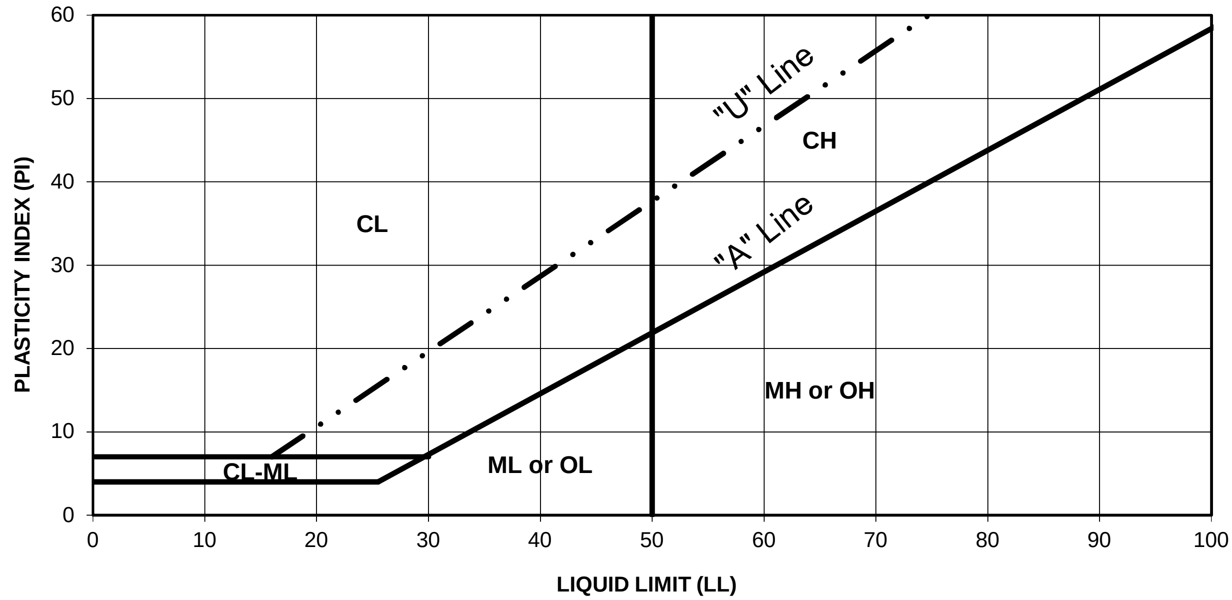
ATTERBERG LIMITS
ASTM D 4318

Client Name:	HDR	Tested By:	SM	Date:	12/03/19
Project Name:	CTO-48 SCORE Ventura Corridor	Computed By:	NR	Date:	12/09/19
Project No.:	10193167	Checked By:	AP	Date:	12/11/19



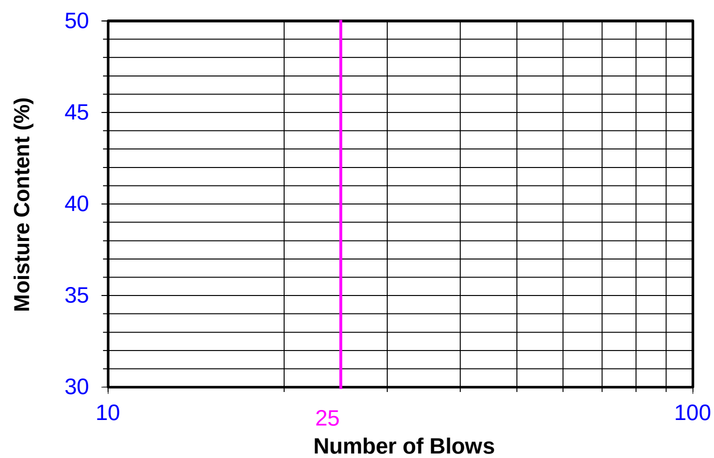
PROCEDURE USED

☐ Wet Preparation

☒ Dry Preparation

☒ Procedure A
Multipoint Test

☐ Procedure B
One-point Test



Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
	A-19-003	2	10	NP	NP	NP	

* NP denotes "non-plastic"

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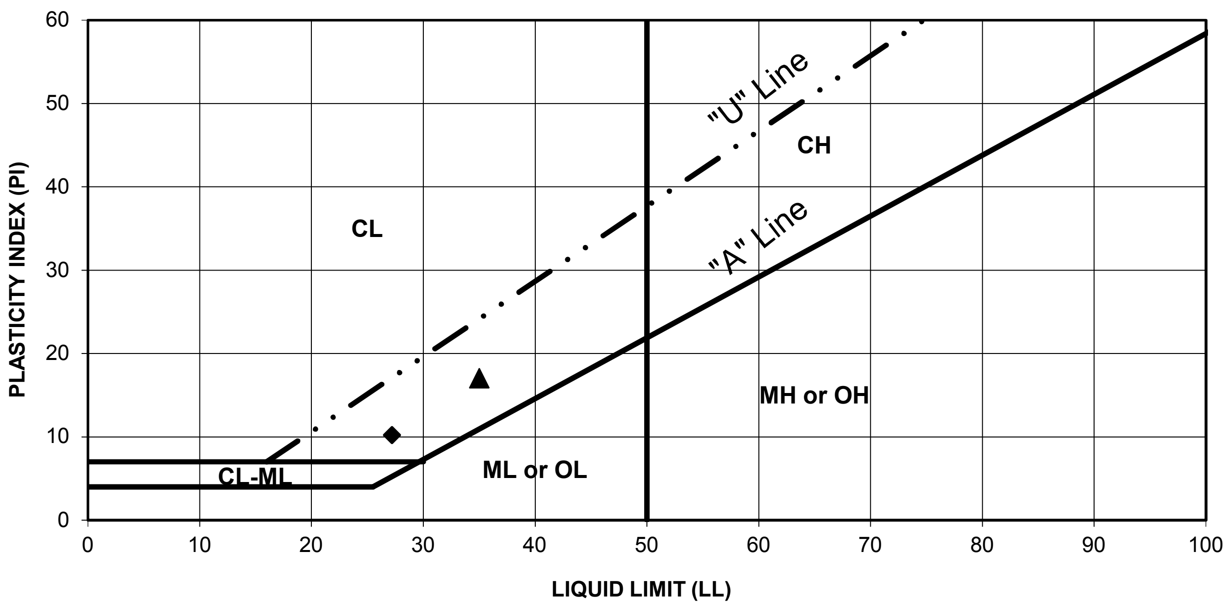
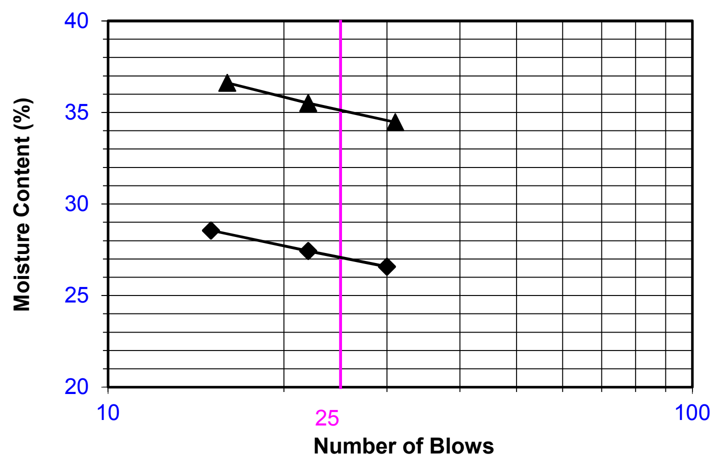
ATTERBERG LIMITS
ASTM D 4318

Client Name: HDR Tested By: SM Date: 12/03/19
Project Name: CTO-48 SCORE Ventura Corridor Computed By: NR Date: 12/09/19
Project No.: 10193167 Checked By: AP Date: 12/11/19



PROCEDURE USED

- [ ] Wet Preparation
- [x] Dry Preparation
- [x] Procedure A
  Multipoint Test
- [ ] Procedure B
  One-point Test



Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	A-19-005	2	5	27	17	10	CL
▲	A-19-005	4	15	35	18	17	CL

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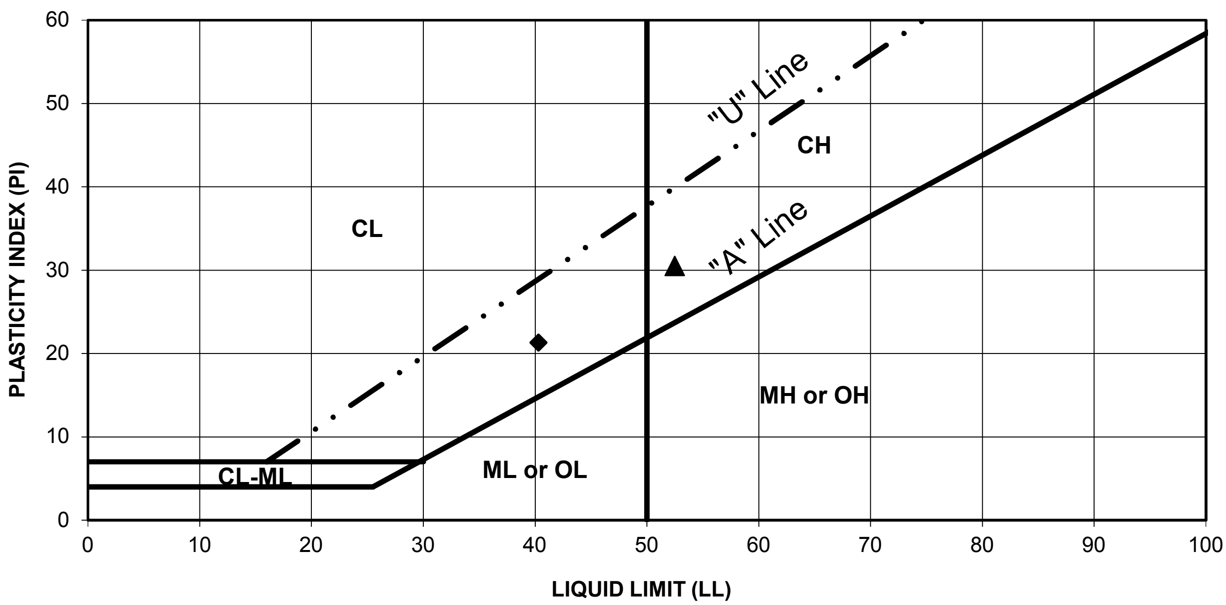
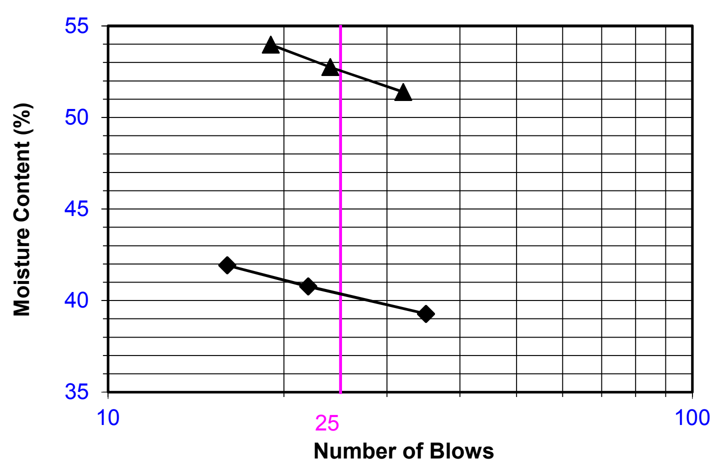
ATTERBERG LIMITS
ASTM D 4318

Client Name:	HDR	Tested By:	SM	Date:	12/03/19
Project Name:	CTO-48 SCORE Ventura Corridor	Computed By:	NR	Date:	12/09/19
Project No.:	10193167	Checked By:	AP	Date:	12/11/19



PROCEDURE USED

- [ ] Wet Preparation
- [X] Dry Preparation
- [X] Procedure A
  Multipoint Test
- [ ] Procedure B
  One-point Test



Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
◆	A-19-005	5	20	40	19	21	CL
▲	A-19-005	7	30	53	22	31	CH

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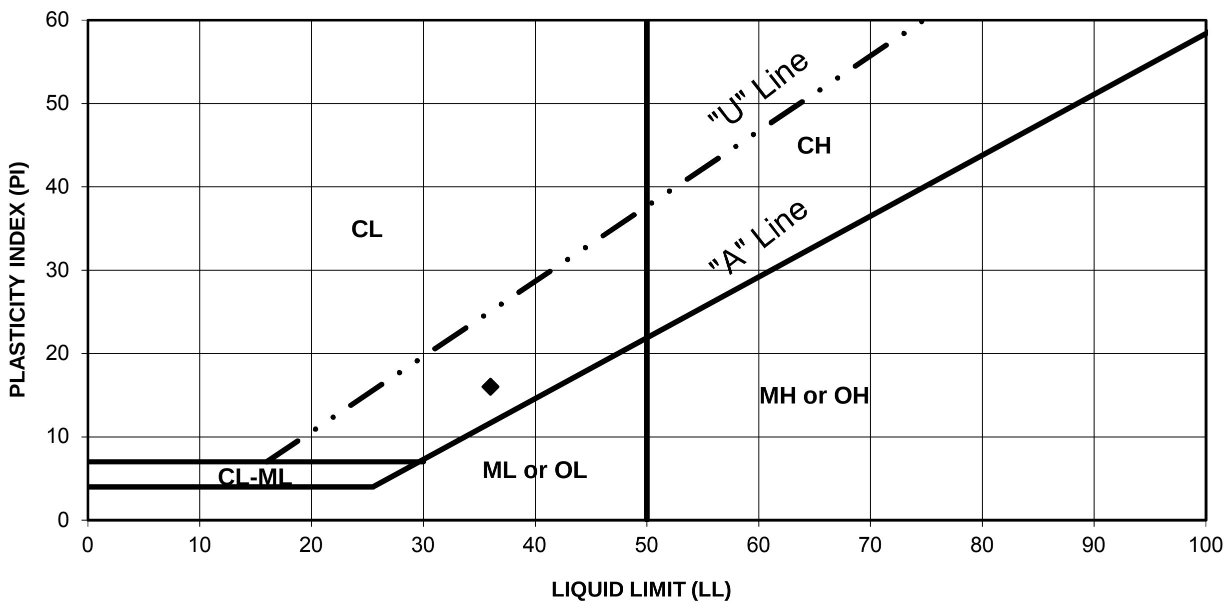
ATTERBERG LIMITS
ASTM D 4318

Client Name:	HDR	Tested By:	SM	Date:	12/03/19
Project Name:	CTO-48 SCORE Ventura Corridor	Computed By:	NR	Date:	12/09/19
Project No.:	10193167	Checked By:	AP	Date:	12/11/19



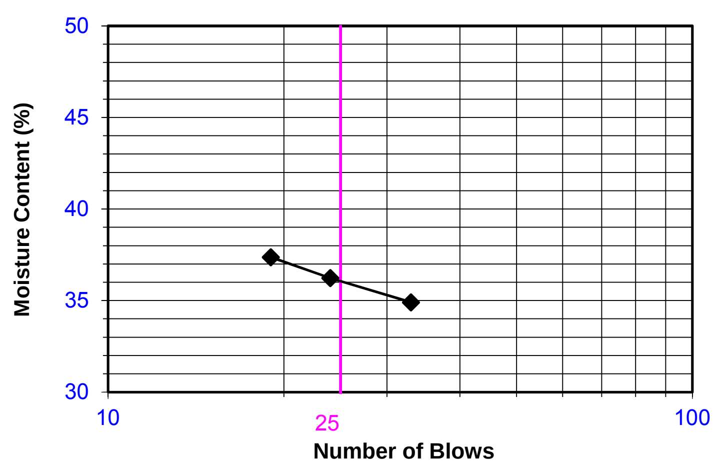
PROCEDURE USED

☐ Wet Preparation

☒ Dry Preparation

☒ Procedure A
Multipoint Test

☐ Procedure B
One-point Test



Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	A-19-006	2	5	36	20	16	CL

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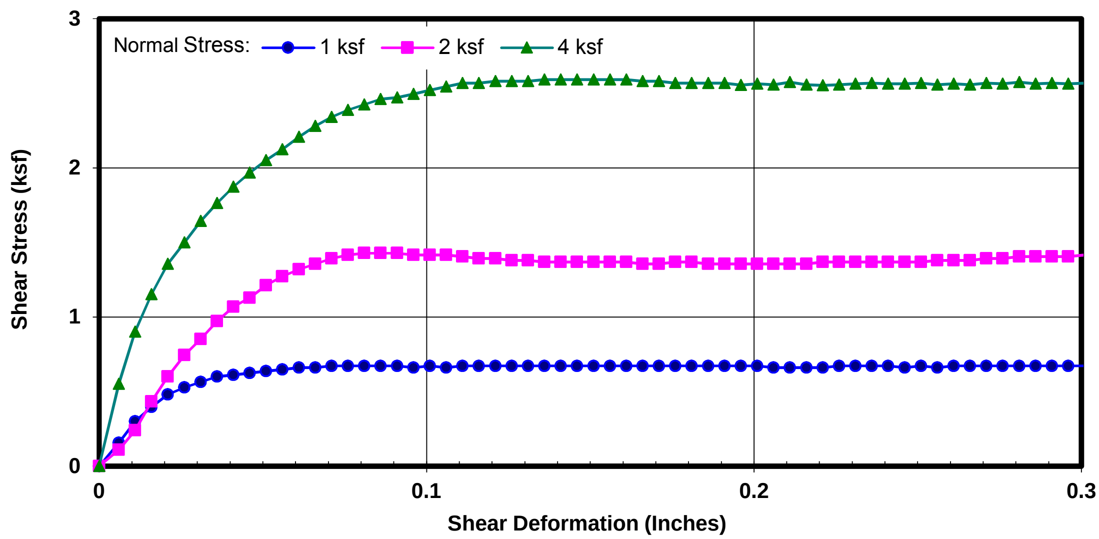
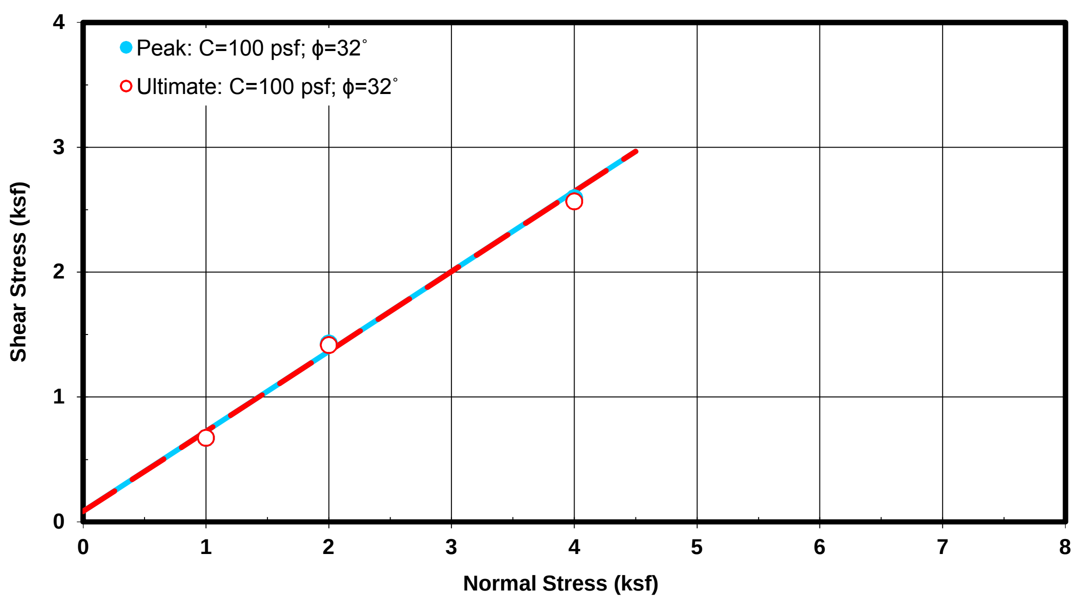
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name: CTO-48 SCORE Ventura Corridor
Project No.: 10193167
Boring No.: A-19-003
Sample No.: 1 **Depth (ft):** 5
Sample Type: Mod. Cal.
Soil Description: Silty Sand
Test Condition: Inundated **Shear Type:** Regular

Tested By: ST **Date:** 12/02/19
Computed By: NR **Date:** 12/05/19
Checked by: AP **Date:** 12/13/19

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
107.7	99.3	8.5	23.9	33	92	1	0.672	0.672
						2	1.428	1.416
						4	2.592	2.568





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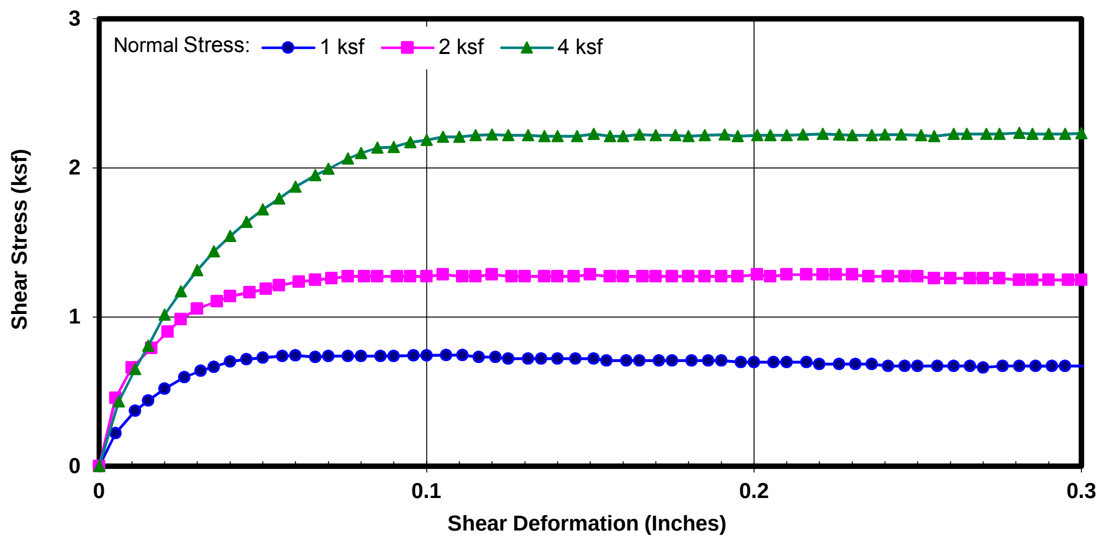
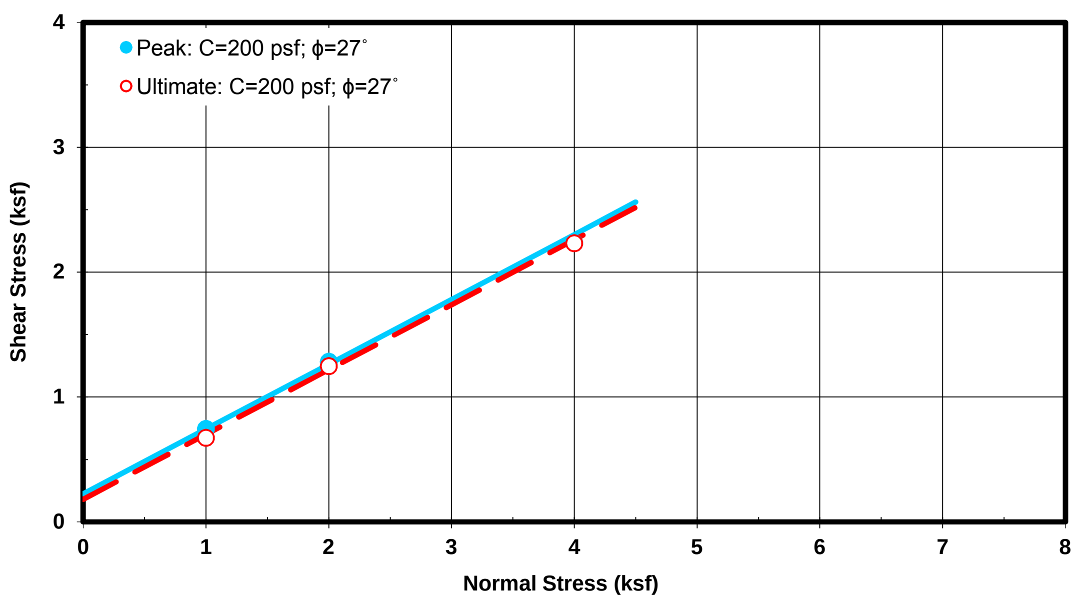
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name: CTO-48 SCORE Ventura Corridor
Project No.: 10193167
Boring No.: A-19-005
Sample No.: 2 **Depth (ft):** 5
Sample Type: Mod. Cal.
Soil Description: Sandy Lean Clay
Test Condition: Inundated **Shear Type:** Regular

Tested By: JT **Date:** 12/04/19
Computed By: NR **Date:** 12/09/19
Checked by: AP **Date:** 12/13/19

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
118.3	101.9	16.1	22.1	66	91	1	0.744	0.672
						2	1.284	1.248
						4	2.234	2.232





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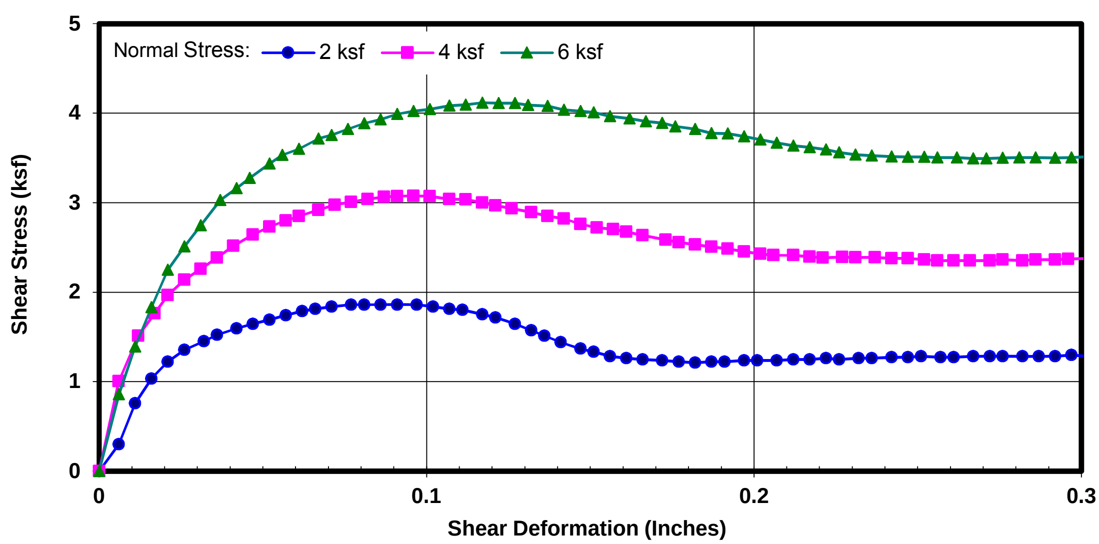
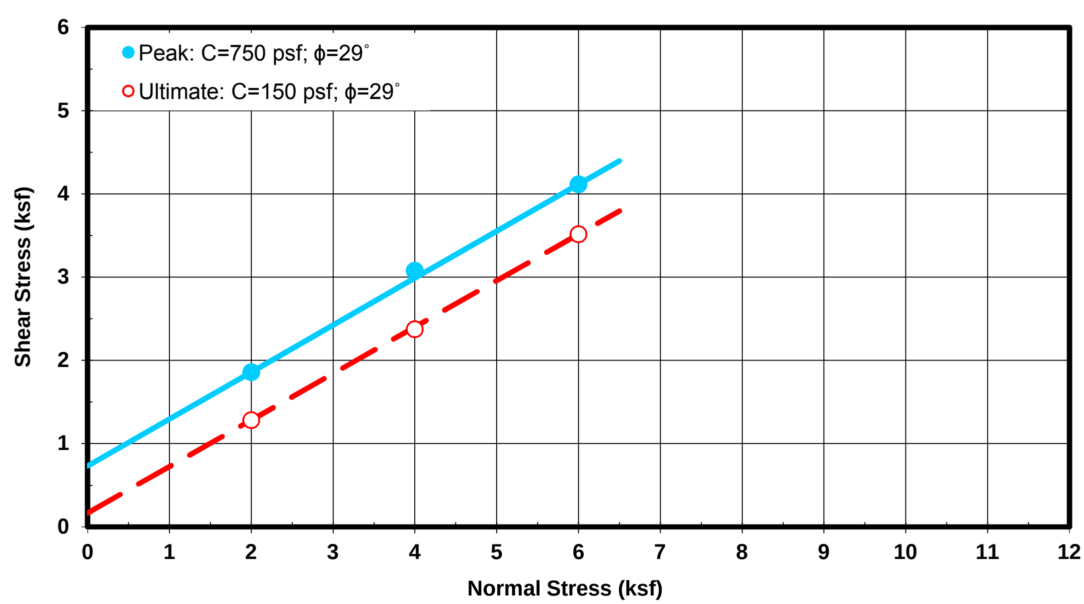
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name: CTO-48 SCORE Ventura Corridor
Project No.: 10193167
Boring No.: A-19-005
Sample No.: 6 **Depth (ft):** 25
Sample Type: Mod. Cal.
Soil Description: Clayey Sand
Test Condition: Inundated **Shear Type:** Regular

Tested By: JT **Date:** 12/04/19
Computed By: NR **Date:** 12/09/19
Checked by: AP **Date:** 12/13/19

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
129.5	107.4	20.6	21.1	98	100	2	1.860	1.284
						4	3.076	2.376
						6	4.117	3.516





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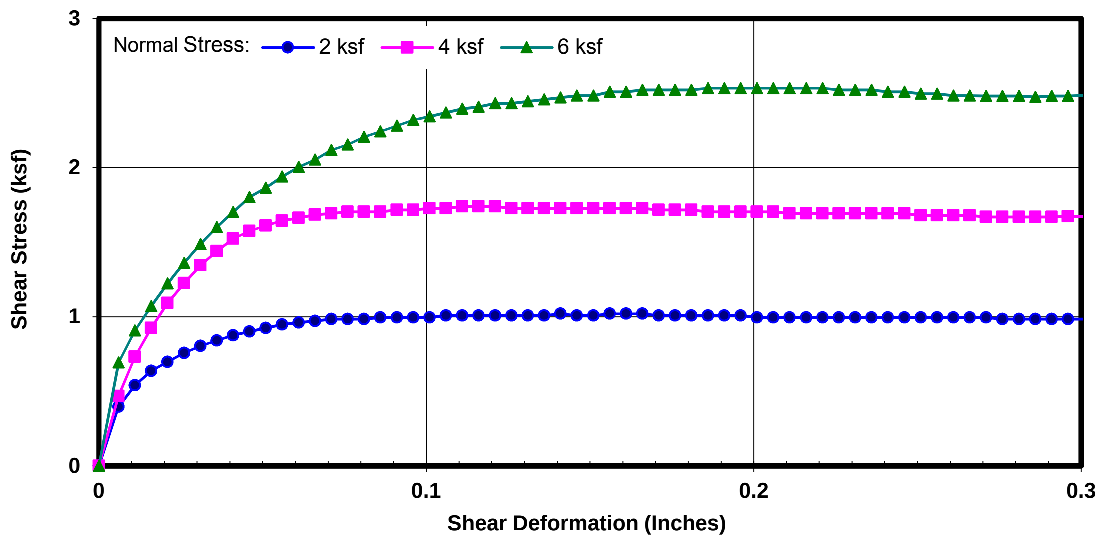
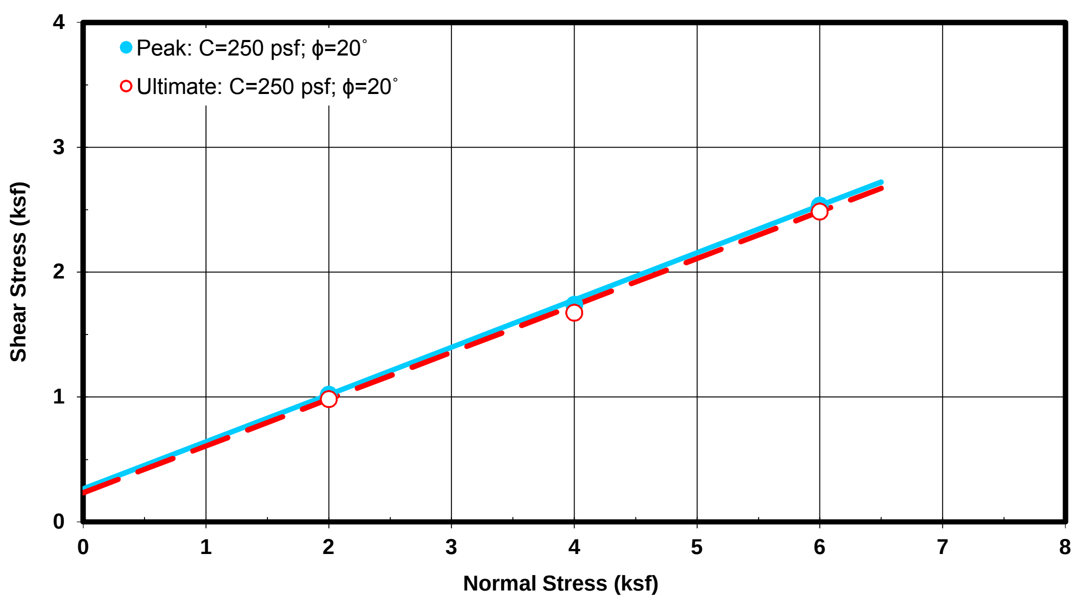
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name: CTO-48 SCORE Ventura Corridor
Project No.: 10193167
Boring No.: A-19-006
Sample No.: 4 **Depth (ft):** 15
Sample Type: Mod. Cal.
Soil Description: Clay
Test Condition: Inundated **Shear Type:** Regular

Tested By: JT **Date:** 12/04/19
Computed By: NR **Date:** 12/09/19
Checked by: AP **Date:** 12/13/19

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
115.0	86.4	33.1	35.1	94	100	2	1.020	0.984
						4	1.740	1.674
						6	2.533	2.485





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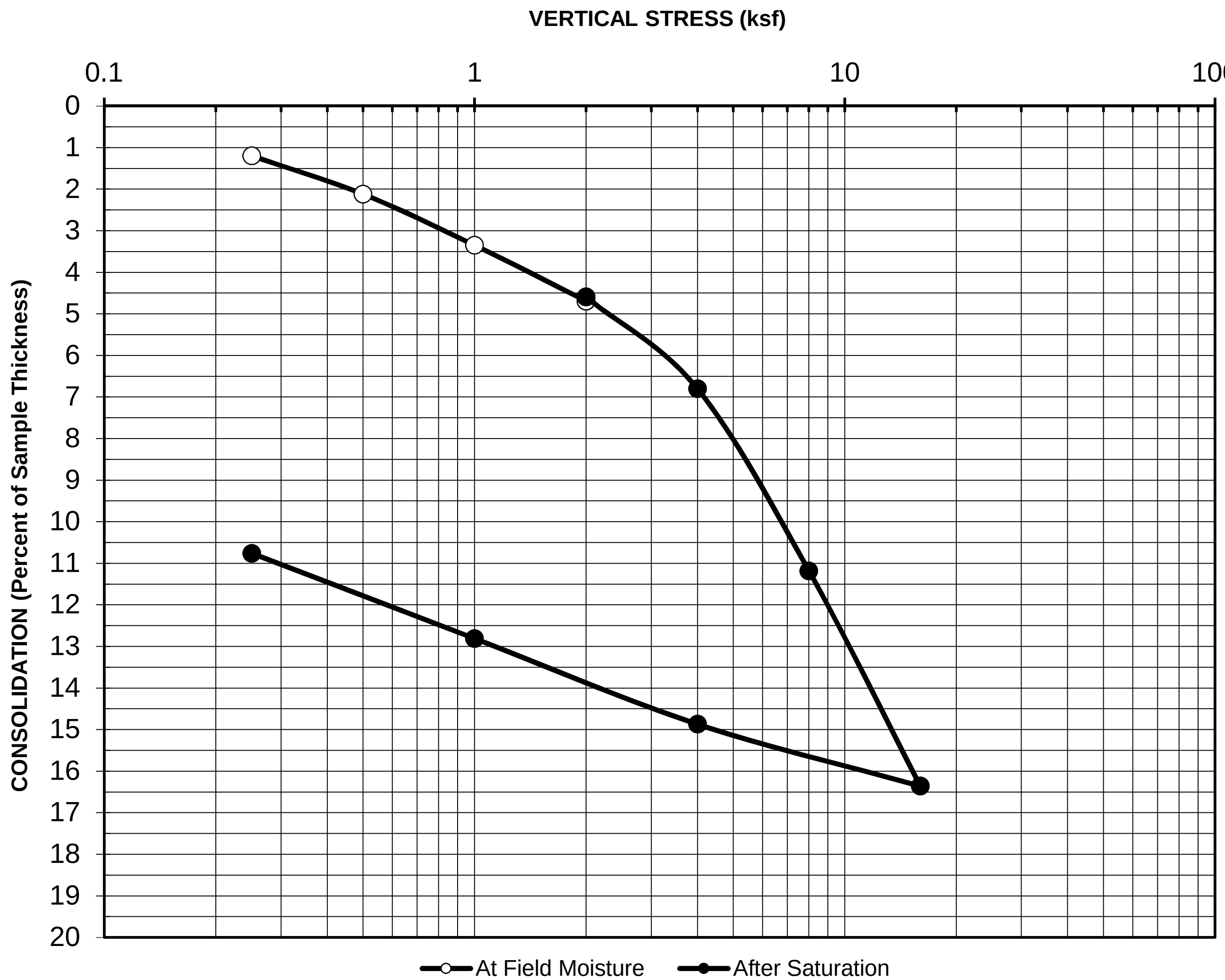

Boring No. :	A-19-005	Initial Dry Unit Weight (pcf):	85.1
Sample No.:	4	Initial Moisture Content (%):	32.1
Depth (feet):	15	Final Moisture Content (%):	34.1
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.7
Soil Description:	Lean Clay w/sand	Initial Void Ratio:	0.98
Remarks:	Swell= 0.10% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

Project Name: CTO-48 SCORE Ventura Corridor
Project No.: 10193167
Date: 11/27/2019
AP No: 19-1164 **Figure No:** 1

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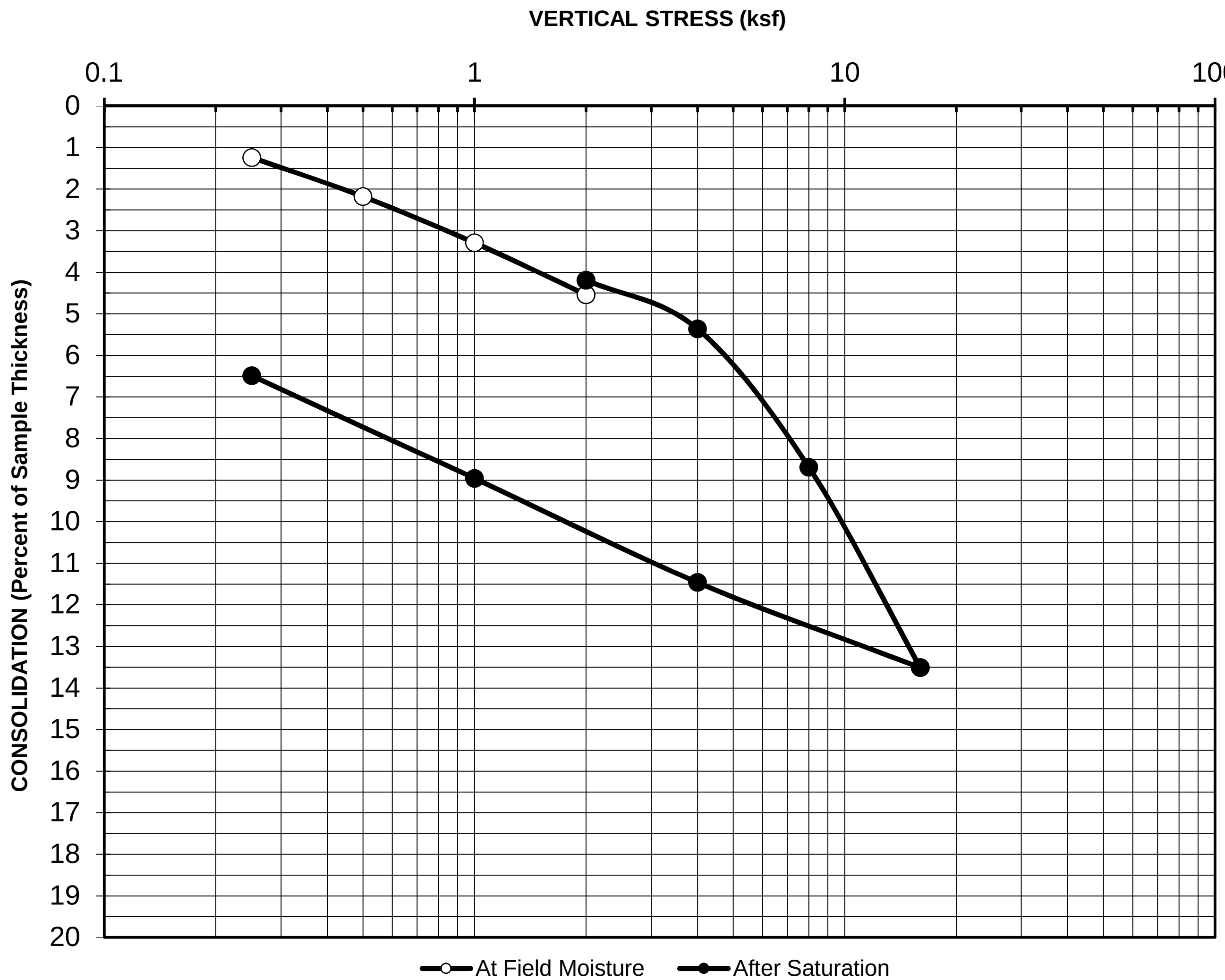

Boring No. :	A-19-005	Initial Dry Unit Weight (pcf):	80.9
Sample No.:	8	Initial Moisture Content (%):	41.0
Depth (feet):	35	Final Moisture Content (%):	38.5
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.7
Soil Description:	Clay	Initial Void Ratio:	1.08
Remarks:	Swell= 0.35% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

Project Name: CTO-48 SCORE Ventura Corridor
Project No.: 10193167
Date: 11/27/2019
AP No: 19-1164 **Figure No:** 1

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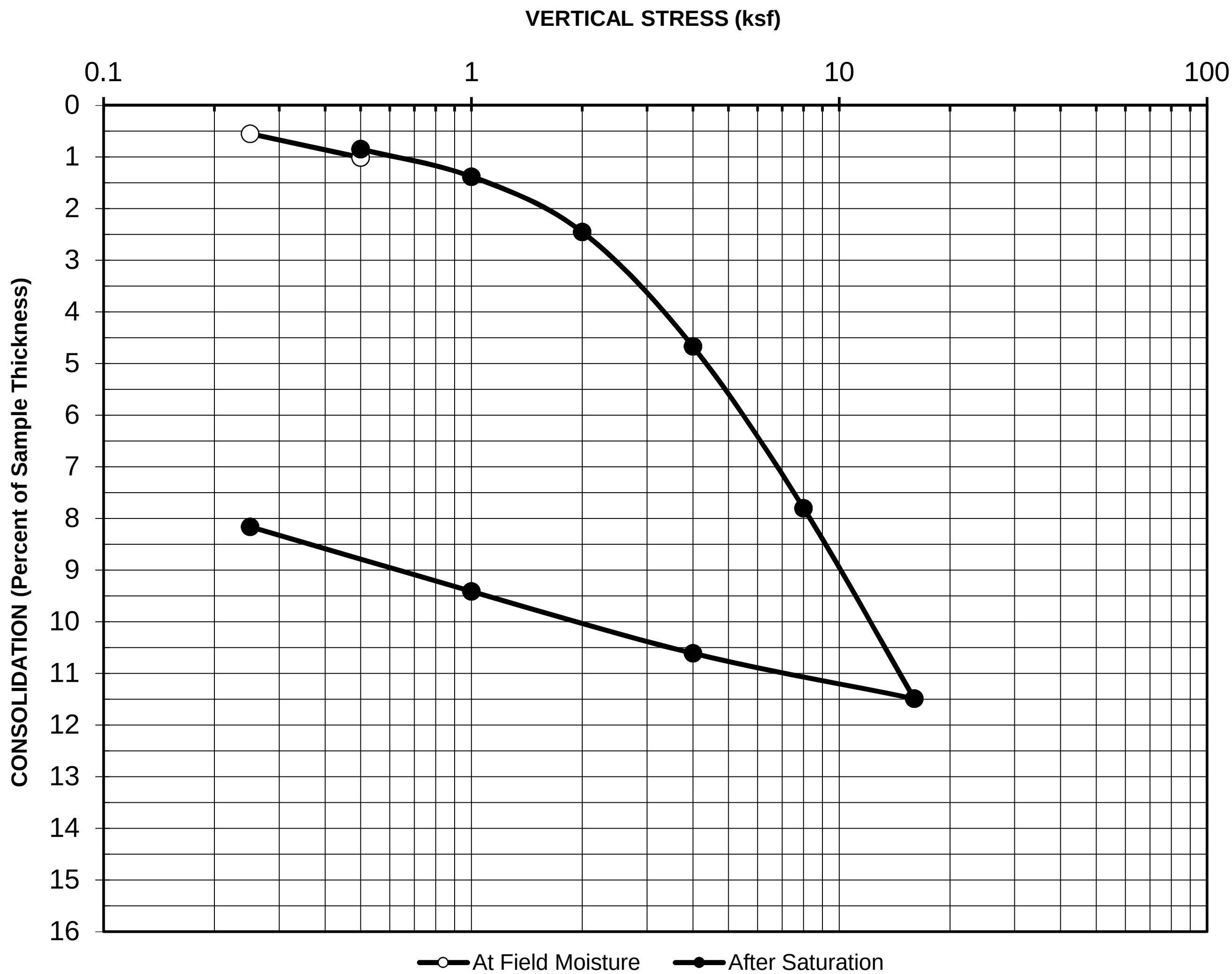

Boring No. :	A-19-006	Initial Dry Unit Weight (pcf):	95.8
Sample No.:	2	Initial Moisture Content (%):	21.2
Depth (feet):	5	Final Moisture Content (%):	22.5
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.7
Soil Description:	Lean Clay w/sand	Initial Void Ratio:	0.76
Remarks:	Swell= 0.16% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

Project Name: CTO-48 SCORE Ventura Corridor
Project No.: 10193167
Date: 11/27/2019
AP No: 19-1164 **Figure No:** 1

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EXPANSION INDEX TEST RESULTS

ASTM D 4829

Client Name: HDR
Project Name: CTO-48 SCORE Ventura Corridor
Project No.: 10193167

AP Job No.: 19-1164
Date: 12/06/19

Boring No.	Sample No.	Depth (ft)	Soil Description	Molded Dry Density (pcf)	Molded Moisture Content (%)	Init. Degree Saturation (%)	Measured Expansion Index	Corrected Expansion Index
A-19-003	B	0-5	Silty Sand w/gravel	110.9	9.3	48.1	3	2

ASTM EXPANSION CLASSIFICATION

Expansion Index	Classification
0-20	V. Low
21-50	Low
51-90	Medium
91-130	High
>130	V. High

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EXPANSION INDEX TEST RESULTS

ASTM D 4829

Client Name:	HDR	AP Job No.:	19-1164
Project Name:	CTO-48 SCORE Ventura Corridor	Date:	12/06/19
Project No.:	10193167		

Boring No.	Sample No.	Depth (ft)	Soil Description	Molded Dry Density (pcf)	Molded Moisture Content (%)	Init. Degree Saturation (%)	Measured Expansion Index	Corrected Expansion Index
A-19-005	B	0-5	Sandy Clay	101.1	12.4	50.2	109	109

ASTM EXPANSION CLASSIFICATION

Expansion Index	Classification
0-20	V. Low
21-50	Low
51-90	Medium
91-130	High
>130	V. High

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R-VALUE TEST DATA

ASTM D2844

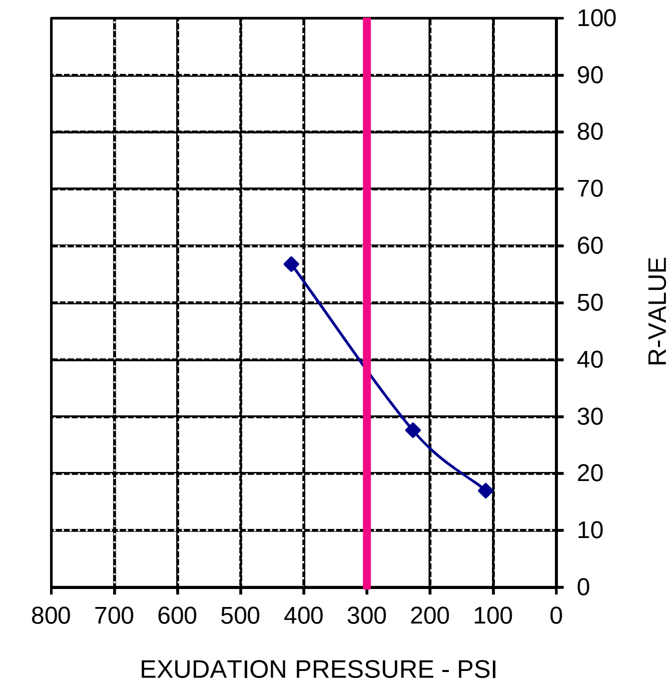
Project Name:	CTO-48 SCORE Ventura Corridor	Tested By:	ST	Date:	12/08/19
Project Number:	10193167	Computed By:	KM	Date:	12/12/19
Boring No.:	A-19-001	Checked By:	AP	Date:	12/13/19
Sample No.:	B	Depth (ft.):	0-5		
Location:	N/A				
Soil Description:	Sandy Silt				

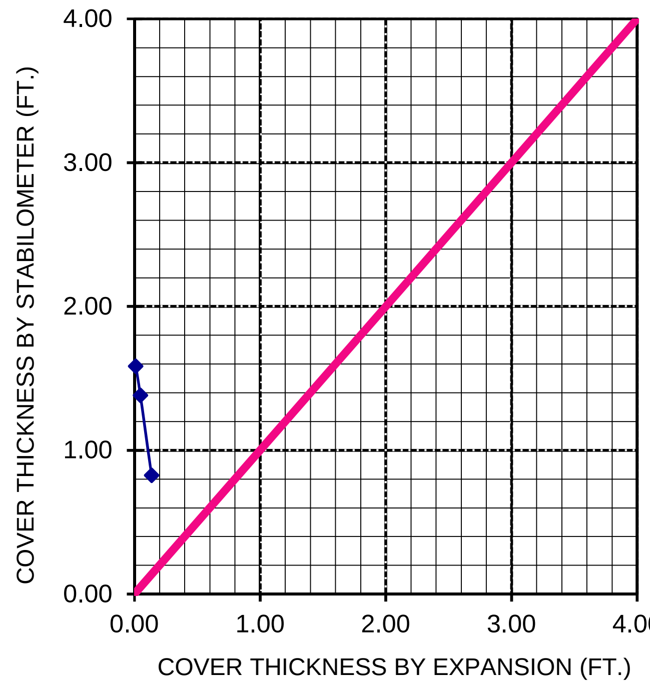
Mold Number	A	B	C	
Water Added, g	32	16	0	
Compact Moisture(%)	19.4	17.6	15.8	
Compaction Gage Pressure, psi	100	250	350	
Exudation Pressure, psi	112	227	420	
Sample Height, Inches	2.6	2.6	2.5	
Gross Weight Mold, g	3030	3026	3012	
Tare Weight Mold, g	1966	1966	1967	
Net Sample Weight, g	1064	1061	1044	
Expansion, inchesx10^{-4}	3	14	41	
Stability 2,000 (160 psi)	53/120	46/102	26/53	
Turns Displacement	4.43	4.07	3.84	
R-Value Uncorrected	16	26	57	
R-Value Corrected	17	28	57	
Dry Density, pcf	103.8	105.2	109.3	
Traffic Index	8.0	8.0	8.0	
G.E. by Stability	1.59	1.38	0.83	
G.E. by Expansion	0.01	0.05	0.14	

R-VALUE		
	By Exudation:	38
	By Expansion:	*N/A
	At Equilibrium: (by Exudation)	38

Remarks

Gf = 1.34, and 0.1 % Retained on the ¾"
*Not Applicable





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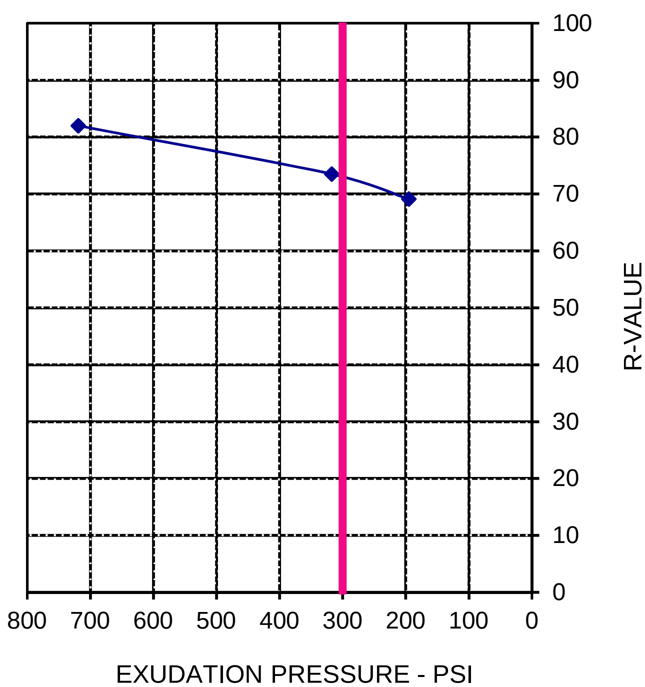
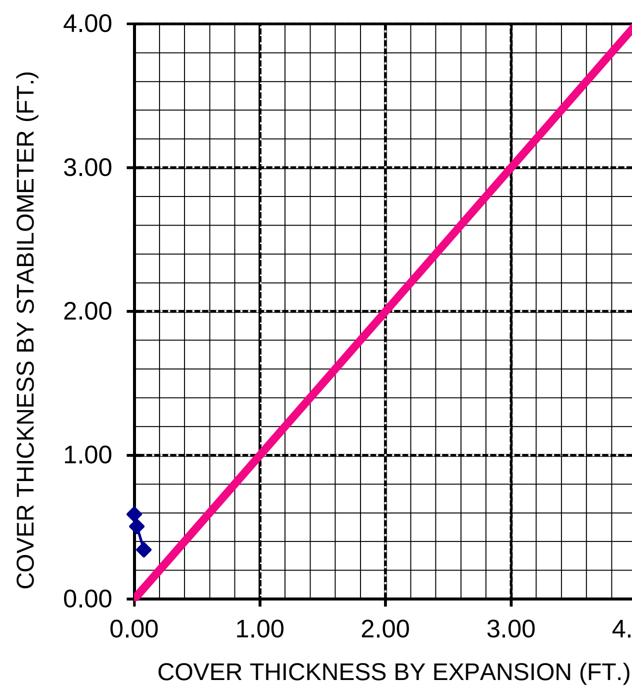
R-VALUE TEST DATA
ASTM D2844

Project Name:	CTO-48 SCORE Ventura Corridor	Tested By:	ST	Date:	12/08/19
Project Number:	10193167	Computed By:	KM	Date:	12/10/19
Boring No.:	A-19-003	Checked By:	AP	Date:	12/13/19
Sample No.:	B	Depth (ft.):	0-5		
Location:	N/A				
Soil Description:	Silty Sand w/gravel				

Mold Number	D	E	F	
Water Added, g	41	31	21	
Compact Moisture(%)	12.5	11.3	10.2	
Compaction Gage Pressure, psi	250	350	350	
Exudation Pressure, psi	195	317	719	
Sample Height, Inches	2.4	2.4	2.4	
Gross Weight Mold, g	3000	2980	2884	
Tare Weight Mold, g	1964	1954	1868	
Net Sample Weight, g	1036	1026	1016	
Expansion, inchesx10^{-4}	0	6	23	
Stability 2,000 (160 psi)	17/32	14/25	11/16	
Turns Displacement	4.18	4.50	4.60	
R-Value Uncorrected	71	75	83	
R-Value Corrected	69	74	82	
Dry Density, pcf	116.3	116.4	116.3	
Traffic Index	8.0	8.0	8.0	
G.E. by Stability	0.59	0.51	0.34	
G.E. by Expansion	0.00	0.02	0.08	

R-VALUE		
	By Exudation:	73
	By Expansion:	*N/A
	At Equilibrium: (by Exudation)	73
Remarks	Gf = 1.34, and 9.3 % Retained on the ¾" *Not Applicable	





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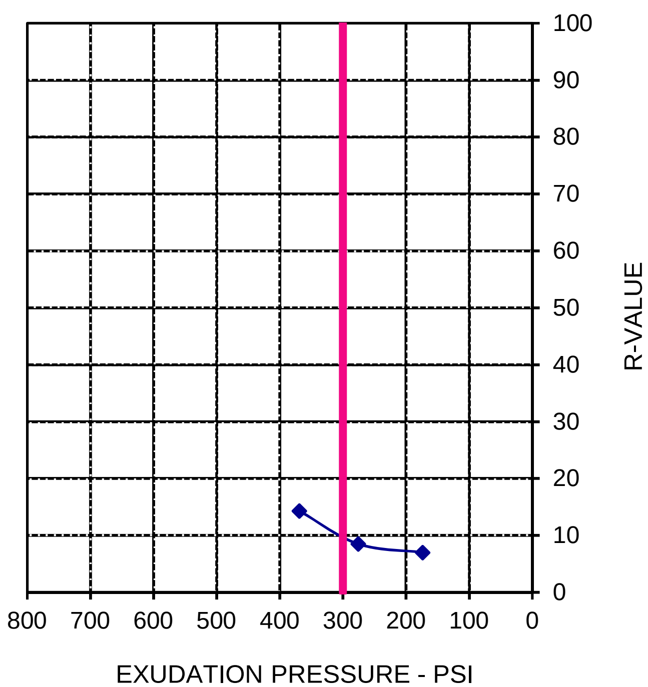
R-VALUE TEST DATA

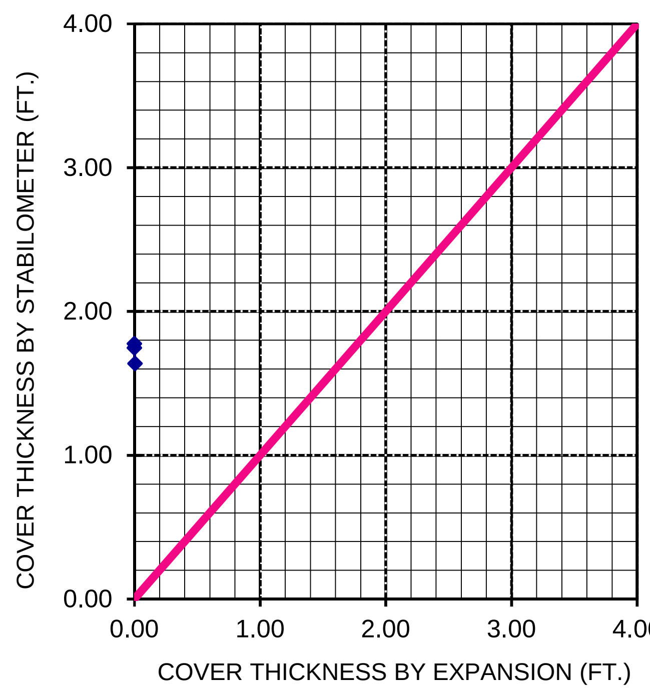
ASTM D2844

Project Name:	CTO-48 SCORE Ventura Corridor	Tested By:	ST	Date:	12/08/19
Project Number:	10193167	Computed By:	KM	Date:	12/12/19
Boring No.:	A-19-005	Checked By:	AP	Date:	12/13/19
Sample Type:	B	Depth (ft.):	0-5		
Location:	N/A				
Soil Description:	Sandy Clay				

Mold Number	D	F	E	
Water Added, g	33	41	51	
Compact Moisture(%)	26.1	27.0	28.1	
Compaction Gage Pressure, psi	50	50	50	
Exudation Pressure, psi	369	276	174	
Sample Height, Inches	2.7	2.6	2.7	
Gross Weight Mold, g	3022	2948	3046	
Tare Weight Mold, g	1964	1868	1954	
Net Sample Weight, g	1059	1080	1092	
Expansion, inchesx10^{-4}	1	0	0	
Stability 2,000 (160 psi)	60/128	64/141	66/145	
Turns Displacement	4.02	4.14	4.42	
R-Value Uncorrected	13	8	6	
R-Value Corrected	14	9	7	
Dry Density, pcf	94.2	99.1	95.7	
Traffic Index	8.0	8.0	8.0	
G.E. by Stability	1.64	1.75	1.78	
G.E. by Expansion	0.00	0.00	0.00	

R-VALUE	By Exudation:	10
	By Expansion:	*N/A
	At Equilibrium: (by Exudation)	10
Remarks	Gf = 1.34, and 1.3 % Retained on the ¾" *Not Applicable	





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ASTM D 2419
SAND EQUIVALENT TEST

Client Name: HDR
Project Name: CTO-48 SCORE Ventura Corridor
Project No.: 10193167

AP Job No.: 19-1164
Test Date: 12/09/19

Boring No.	Sample No.	Depth (feet)	Soil Description	Clay Reading	Sand Reading	Corrected Sand Reading	Sand Equivalent
A-19-003	B	0-5	Silty Sand w/gravel	11.7	11.6	1.6	14

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COMPACTION TEST

Client: HDR | AP Number: 19-1164
Project Name: CTO-48 SCORE Ventura Corridor | Tested By: SM | Date: 12/09/19
Project No. : 10193167 | Calculated By: NR | Date: 12/12/19
Boring No.: A-19-001 | Checked By: AP | Date: 12/13/19
Sample No.: B | Depth(ft.): 0-5
Visual Sample Description: Sandy Silt

Compaction Method: [X] ASTM D1557 [ ] ASTM D698
Preparation Method: [ ] Moist [X] Dry

METHOD: A
MOLD VOLUME (CU.FT): 0.0333

Wt. Comp. Soil + Mold (gm.)	3839	3778	3824	3716		
Wt. of Mold (gm.)	1853	1853	1853	1853		
Net Wt. of Soil (gm.)	1986	1925	1971	1863		
Container No.						
Wt. of Container (gm.)	143.36	150.80	263.11	235.85		
Wet Wt. of Soil + Cont. (gm.)	533.58	413.09	722.05	578.62		
Dry Wt. of Soil + Cont. (gm.)	478.28	370.88	665.54	543.89		
Moisture Content (%)	16.51	19.18	14.04	11.27		
Wet Density (pcf)	131.32	127.28	130.32	123.18		
Dry Density (pcf)	112.71	106.80	114.28	110.70		

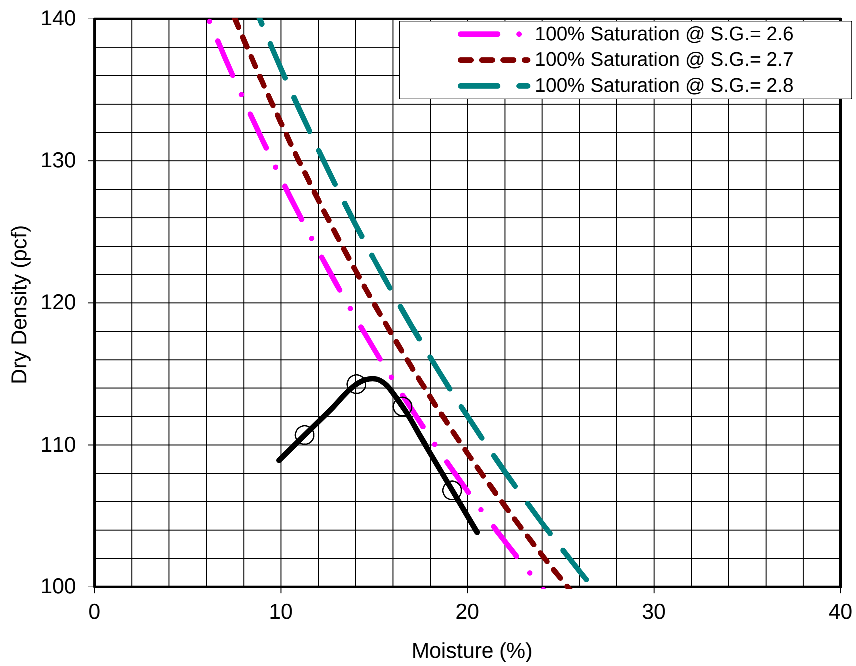
Maximum Dry Density (pcf): **114.6**
Maximum Dry Density w/ Rock Correction (pcf): **N/A**
Optimum Moisture Content (%): **14.9**
Optimum Moisture Content w/ Rock Correction (%): **N/A**

PROCEDURE USED

[X] *METHOD A: Percent of Oversize:* *2.1%*
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)

[ ] *METHOD B: Percent of Oversize:* *N/A*
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)

[ ] *METHOD C: Percent of Oversize:* *N/A*
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)



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COMPACTION TEST

Client:	HDR			AP Number:	19-1164
Project Name:	CTO-48 SCORE Ventura Corridor	Tested By:	JT	Date:	12/09/19
Project No. :	10193167	Calculated By:	KM	Date:	12/12/19
Boring No.:	A-19-003	Checked By:	AP	Date:	12/13/19
Sample No.:	B	Depth(ft.):	0-5		

Visual Sample Description: Silty Sand w/gravel

Compaction Method: [X] ASTM D1557 [ ] ASTM D698

Preparation Method: [ ] Moist [X] Dry

METHOD: A
MOLD VOLUME (CU.FT): 0.0333

Wt. Comp. Soil + Mold (gm.)	3904	3838	3701	3846		
Wt. of Mold (gm.)	1854	1854	1854	1854		
Net Wt. of Soil (gm.)	2050	1984	1847	1992		
Container No.						
Wt. of Container (gm.)	148.45	141.42	150.07	148.27		
Wet Wt. of Soil + Cont. (gm.)	554.53	546.92	556.80	555.02		
Dry Wt. of Soil + Cont. (gm.)	510.11	495.27	529.59	518.80		
Moisture Content (%)	12.28	14.60	7.17	9.78		
Wet Density (pcf)	135.58	131.22	122.12	131.75		
Dry Density (pcf)	120.75	114.50	113.95	120.01		

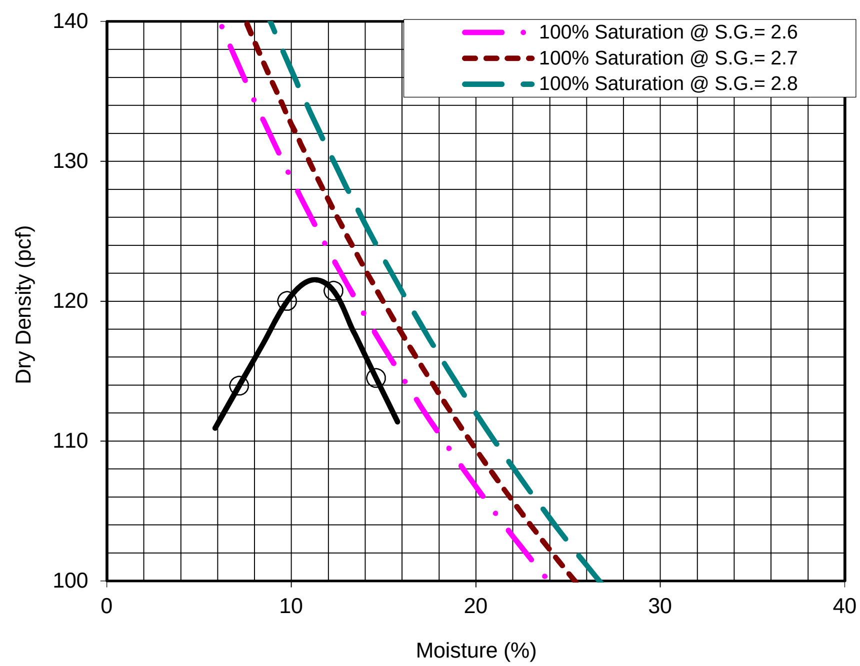
Maximum Dry Density (pcf)	121.6	Optimum Moisture Content (%)	11.1
Maximum Dry Density w/ Rock Correction (pcf)	129.5	Optimum Moisture Content w/ Rock Correction (%)	8.7

PROCEDURE USED

[X] *METHOD A: Percent of Oversize:* *22.0%*
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)

[ ] *METHOD B: Percent of Oversize:* N/A
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)

[ ] *METHOD C: Percent of Oversize:* N/A
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)



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COMPACTION TEST

Client:	HDR			AP Number:	19-1164
Project Name:	CTO-48 SCORE Ventura Corridor	Tested By:	SM	Date:	12/09/19
Project No. :	10193167	Calculated By:	NR	Date:	12/12/19
Boring No.:	A-19-005	Checked By:	AP	Date:	12/13/19
Sample No.:	B	Depth(ft.):	0-5		

Visual Sample Description: Sandy Clay

Compaction Method: [X] ASTM D1557 [ ] ASTM D698

Preparation Method: [ ] Moist [X] Dry

METHOD: A
MOLD VOLUME (CU.FT): 0.0333

Wt. Comp. Soil + Mold (gm.)	3833	3770	3811	3675		
Wt. of Mold (gm.)	1853	1853	1853	1853		
Net Wt. of Soil (gm.)	1980	1917	1958	1822		
Container No.						
Wt. of Container (gm.)	149.47	147.50	240.00	232.96		
Wet Wt. of Soil + Cont. (gm.)	645.53	457.56	644.49	433.44		
Dry Wt. of Soil + Cont. (gm.)	573.98	405.09	594.97	413.79		
Moisture Content (%)	16.85	20.37	13.95	10.87		
Wet Density (pcf)	130.92	126.79	129.46	120.50		
Dry Density (pcf)	112.04	105.33	113.61	108.69		

Maximum Dry Density (pcf)	**114.1**	Optimum Moisture Content (%)	**15.0**
Maximum Dry Density w/ Rock Correction (pcf)	**116.4**	Optimum Moisture Content w/ Rock Correction (%)	**14.1**

PROCEDURE USED

[X] *METHOD A: Percent of Oversize:* *6.2%*
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)

[ ] *METHOD B: Percent of Oversize:* *N/A*
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)

[ ] *METHOD C: Percent of Oversize:* *N/A*
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)

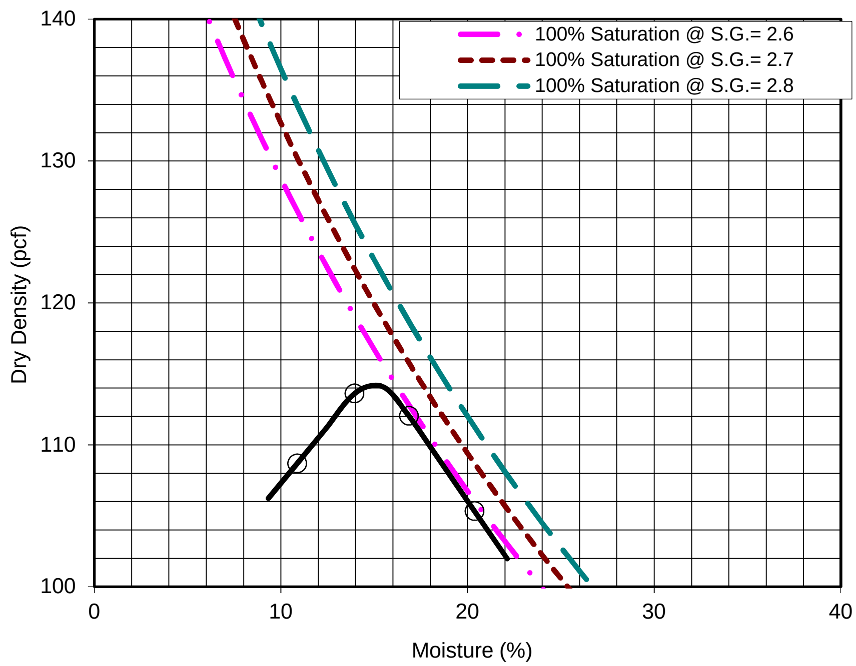


Table 1 - Laboratory Tests on Soil Samples

HDR, Irvine
CTO-48 SCORE Corridor-Simi Valley
Your #10193167, HDR Lab #19-0853LAB
11-Dec-19

Sample ID			A-19-001 @ 3'	A-19-005 @ 3-5'	A-19-005 @ 15-20'
Resistivity		**Units**			
as-received		ohm-cm	3,680	2,640	1,440
minimum		ohm-cm	1,680	1,560	1,440
pH			7.4	7.4	7.6
Electrical Conductivity		mS/cm	0.17	0.18	0.18
Chemical Analyses					
Cations					
calcium	Ca^{2+}	mg/kg	116	143	91
magnesium	Mg^{2+}	mg/kg	20	7.6	16
sodium	Na^{1+}	mg/kg	57	25	95
potassium	K^{1+}	mg/kg	41	35	19
Anions					
carbonate	CO_3^{2-}	mg/kg	ND	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	442	363	442
fluoride	F^{1-}	mg/kg	3.4	3.1	5.5
chloride	Cl^{1-}	mg/kg	0.7	3.1	3.6
sulfate	SO_4^{2-}	mg/kg	47	46	95
phosphate	PO_4^{3-}	mg/kg	9.4	14	ND
Other Tests					
ammonium	NH_4^{1+}	mg/kg	ND	ND	ND
nitrate	NO_3^{1-}	mg/kg	158	220	31
sulfide	S^{2-}	qual	na	na	na
Redox		mV	na	na	na

Minimum resistivity per CTM 643, Chlorides per CTM 422, Sulfates per CTM 417

Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.

mg/kg = milligrams per kilogram (parts per million) of dry soil.

Redox = oxidation-reduction potential in millivolts

ND = not detected

na = not analyzed

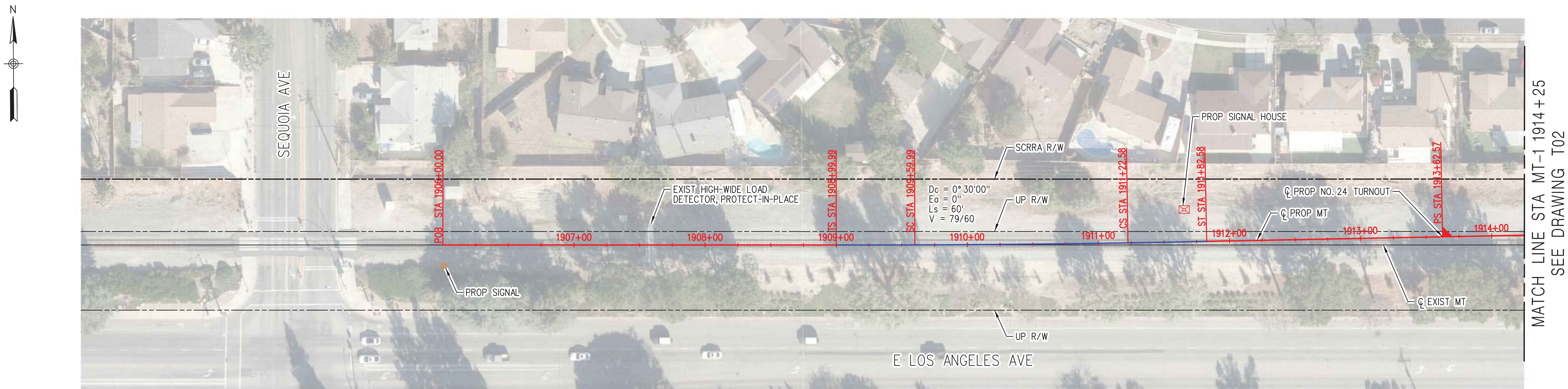
431 West Baseline Road · Claremont, CA 91711
Phone: 909.962.5485 · Fax: 909.626.3316

Appendix D

Proposed Site Improvements

TIMETABLE WEST
TO MOORPARK

TIMETABLE EAST
TO LOS ANGELES

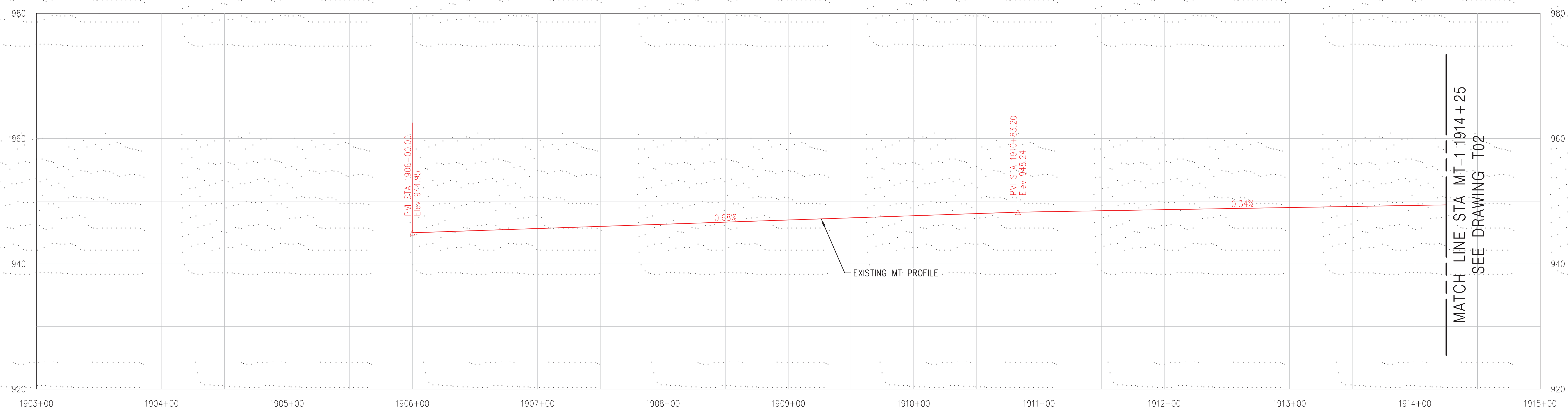


NOTE:
1. EXISTING MT PROFILE FROM SCRRA TRACK CHARTS REPRESENT APPROXIMATE PVI'S AND AVERAGE GRADES
2. TIES, CLIPS, PLATES AND OTM SHALL BE REPLACED WITHIN PROJECT FOOTPRINTS WHERE PROPOSED ALIGNMENT MATCHES EXISTING

100' 0' 100' 200'
GRAPHIC SCALE

LEGEND

- EXIST TRACK
- SCRRA ROW
- UP ROW
- EXIST BRIDGE
- NEW BRIDGE
- EXIST CULVERT
- NEW CULVERT
- NEW TRACK
- SHIFTED TRACK
- NEW CURVE TRACK
- SHIFTED CURVE TRACK
- EXIST GRADE XING PANEL
- NEW GRADE XING PANEL
- NEW RETAINING WALL
- NEW FENCE
- NEW PLATFORM
- NEW SIGNAL



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DRAWN BY	D.RAWN
CHECKED BY	C.HECKER
APPROVED BY	I.NCHARGE
DATE	XX-XX-XXXX

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HDR, Inc.
2280 Market Street, Suite 100
Riverside, CA 92501-2110
(951) 320-7300

SUBMITTED: PROJECT MANAGER

APPROVED:

VENTURA SCORE PHASE I PDR
SIMI VALLEY DOUBLE TRACK AND PLATFORM

ALTERNATIVE 2
SHEET 1 OF 12

CONTRACT NO.	C0000-00
DRAWING NO.	T01
REVISION	SHEET NO. 0000 OF 0000
SCALE	AS NOTED

5/21/2019
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TIMETABLE WEST
TO MOORPARK

TIMETABLE EAST
TO LOS ANGELES

NOTE:
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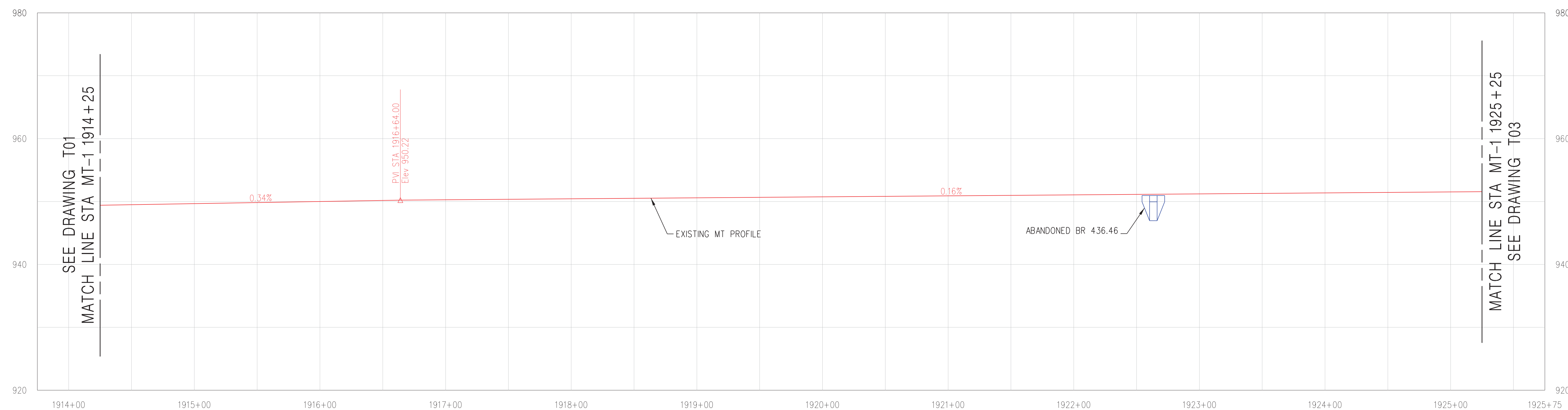
100' 0' 100' 200'
GRAPHIC SCALE

LEGEND
- NEW TRACK
- SHIFTED TRACK
- NEW CURVE TRACK
- SHIFTED CURVE TRACK
- EXIST TRACK
- SCRRA ROW
- UP ROW
- EXIST BRIDGE
- NEW BRIDGE
- EXIST CULVERT
- NEW CULVERT
- EXIST GRADE XING PANEL
- NEW GRADE XING PANEL
- NEW RETAINING WALL
- NEW FENCE
- NEW PLATFORM
- NEW SIGNAL



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APPROVED BY I.NCHARGE
DATE XX-XX-XXXX

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APPROVED:

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ALTERNATIVE 2
SHEET 2 OF 12

CONTRACT NO. C0000-00
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SHEET NO. 0000 OF 0000
SCALE AS NOTED

TIMETABLE WEST
TO MOORPARK

TIMETABLE EAST
TO LOS ANGELES

NOTE:
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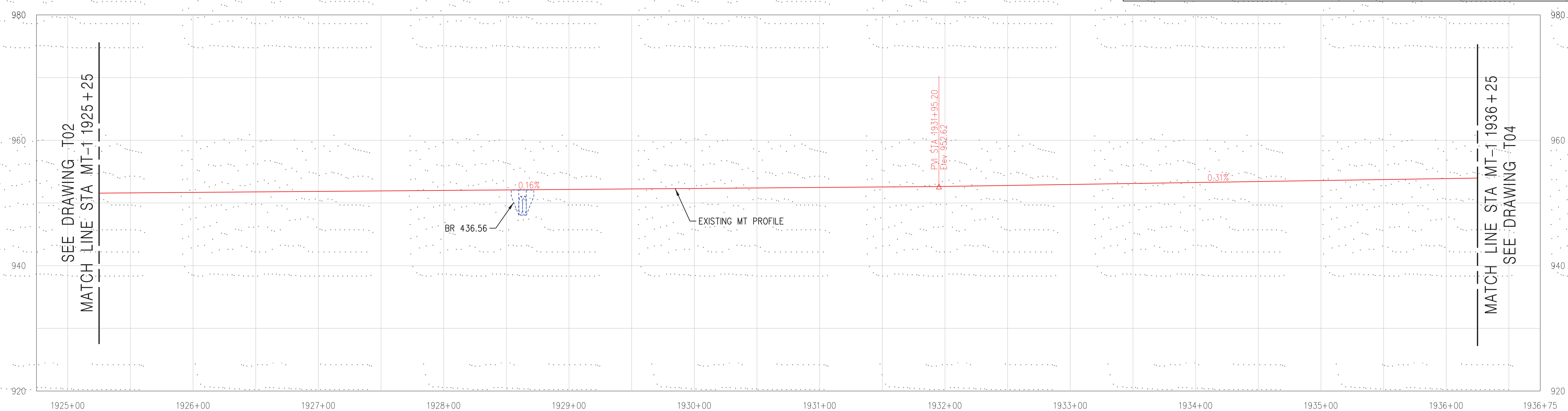
2. TIES, CLIPS, PLATES AND OTM SHALL BE REPLACED WITHIN PROJECT FOOTPRINTS WHERE PROPOSED ALIGNMENT MATCHES EXISTING

100' 0' 100' 200'
GRAPHIC SCALE

LEGEND

- NEW TRACK
- SHIFTED TRACK
- NEW CURVE TRACK
- SHIFTED CURVE TRACK
- EXIST TRACK
- SCRRA ROW
- UP ROW
- EXIST BRIDGE
- NEW BRIDGE
- EXIST CULVERT
- NEW CULVERT
- EXIST GRADE XING PANEL
- NEW GRADE XING PANEL
- NEW RETAINING WALL
- NEW FENCE
- NEW PLATFORM
- NEW SIGNAL



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DATE XX-XX-XXXX

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SUBMITTED: PROJECT MANAGER
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VENTURA SCORE PHASE I PDR
SIMI VALLEY DOUBLE TRACK AND PLATFORM

ALTERNATIVE 2
SHEET 3 OF 12

CONTRACT NO. C0000-00
DRAWING NO. T03
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SHEET NO. 0000 OF 0000
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TIMETABLE WEST
TO MOORPARK

TIMETABLE EAST
TO LOS ANGELES

NOTE:
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2. TIES, CLIPS, PLATES AND OTM SHALL BE REPLACED WITHIN PROJECT FOOTPRINTS WHERE PROPOSED ALIGNMENT MATCHES EXISTING
3. PROPOSED GRADE CROSSING IMPROVEMENT TO BE QUIET ZONE READY

100' 0' 100' 200'
GRAPHIC SCALE

LEGEND
- EXIST TRACK
- SCRRA ROW
- UP ROW
- EXIST BRIDGE
- NEW BRIDGE
- EXIST CULVERT
- NEW CULVERT
- NEW TRACK
- SHIFTED TRACK
- NEW CURVE TRACK
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- EXIST GRADE XING PANEL
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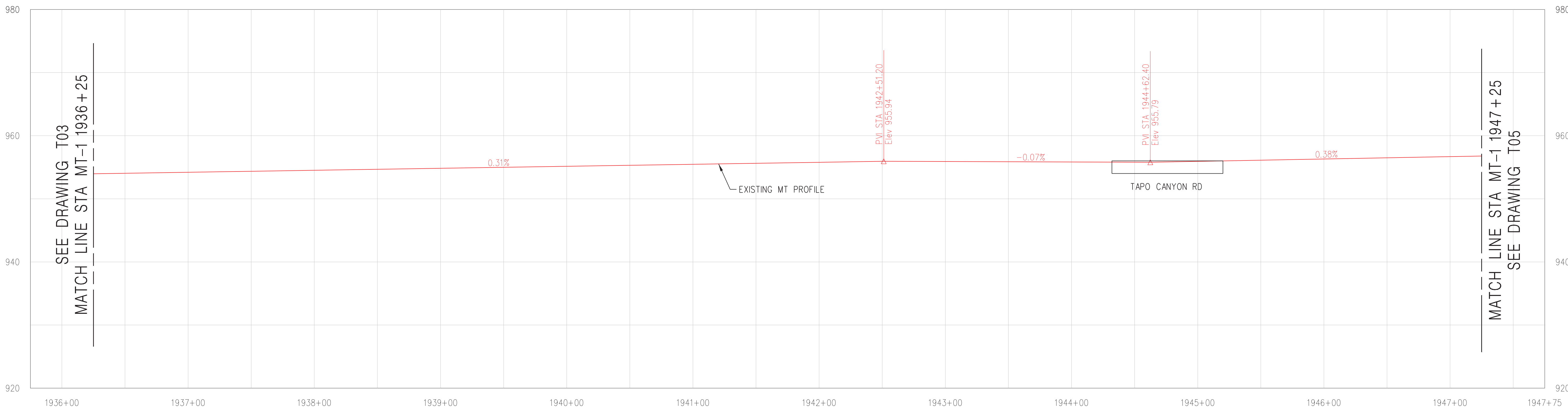
- NEW SIGNAL



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(951) 320-7300

SUBMITTED: PROJECT MANAGER
APPROVED:

VENTURA SCORE PHASE I PDR
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ALTERNATIVE 2
SHEET 4 OF 12

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SHEET NO. 0000 OF 0000
SCALE AS NOTED

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TIMETABLE WEST
TO MOORPARK

TIMETABLE EAST
TO LOS ANGELES

N

SEE DRAWING T04
MATCH LINE STA MT-1 1947+25

MATCH LINE STA MT-1 1958+25
SEE DRAWING T06

NOTE:
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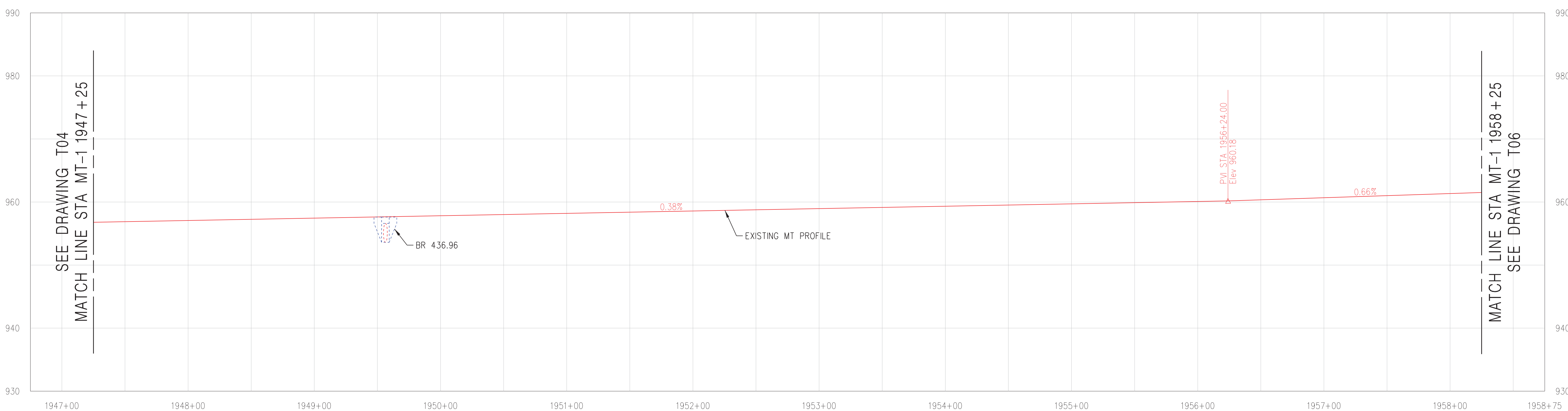
100' 0' 100' 200'
GRAPHIC SCALE

LEGEND

NEW TRACK	EXIST TRACK
SHIFTED TRACK	SCRRA ROW
NEW CURVE TRACK	UP ROW
SHIFTED CURVE TRACK	EXIST BRIDGE
	NEW BRIDGE
	EXIST CULVERT
	NEW CULVERT
	EXIST GRADE XING PANEL
	NEW GRADE XING PANEL
	NEW RETAINING WALL
	NEW FENCE
	NEW PLATFORM
	NEW SIGNAL



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REV.	DATE		BY	SUB.	APP.

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DESIGNED BY: D.ESIGNER
DRAWN BY: D.RAWN
CHECKED BY: C.HECKER
APPROVED BY: I.NCHARGE
DATE: XX-XX-XXXX

METROLINK®

HDR, Inc.
2280 Market Street, Suite 100
Riverside, CA 92501-2110
(951) 320-7300

SUBMITTED: PROJECT MANAGER
APPROVED:

VENTURA SCORE PHASE I PDR
SIMI VALLEY DOUBLE TRACK AND PLATFORM

ALTERNATIVE 2
SHEET 5 OF 12

CONTRACT NO. C0000-00
DRAWING NO. T05
REVISION
SHEET NO. 0000 OF 0000
SCALE AS NOTED

TIMETABLE WEST
TO MOORPARK

TIMETABLE EAST
TO LOS ANGELES

NOTE:
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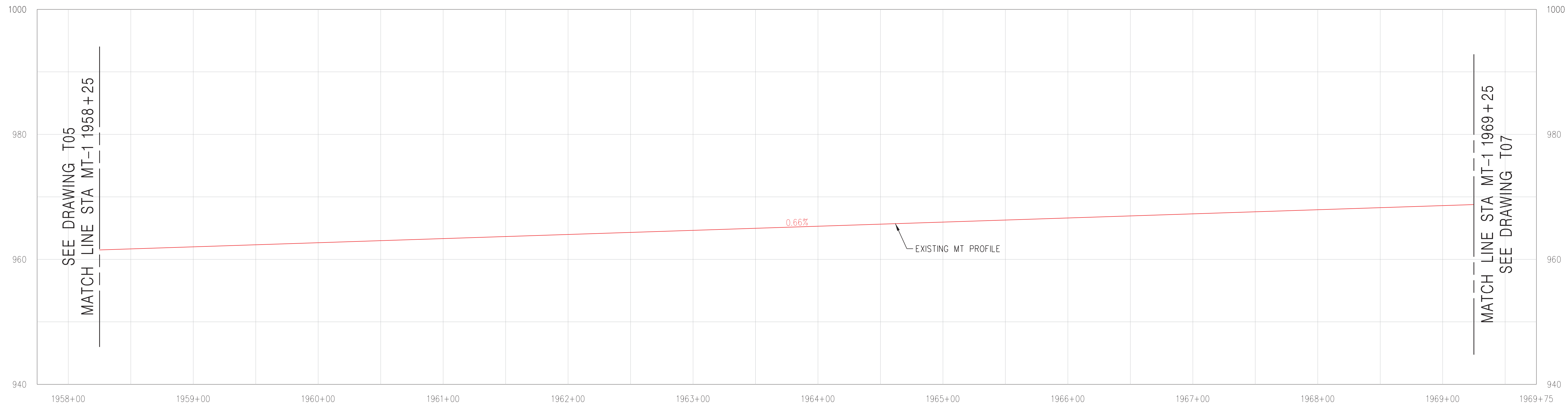
2. TIES, CLIPS, PLATES AND OTM SHALL BE REPLACED WITHIN PROJECT FOOTPRINTS WHERE PROPOSED ALIGNMENT MATCHES EXISTING

100' 0' 100' 200'
GRAPHIC SCALE

LEGEND
- NEW TRACK
- SHIFTED TRACK
- NEW CURVE TRACK
- SHIFTED CURVE TRACK
- EXIST TRACK
- SCRRA ROW
- UP ROW
- EXIST BRIDGE
- NEW BRIDGE
- EXIST CULVERT
- NEW CULVERT
- EXIST GRADE XING PANEL
- NEW GRADE XING PANEL
- NEW RETAINING WALL
- NEW FENCE
- NEW PLATFORM
- NEW SIGNAL



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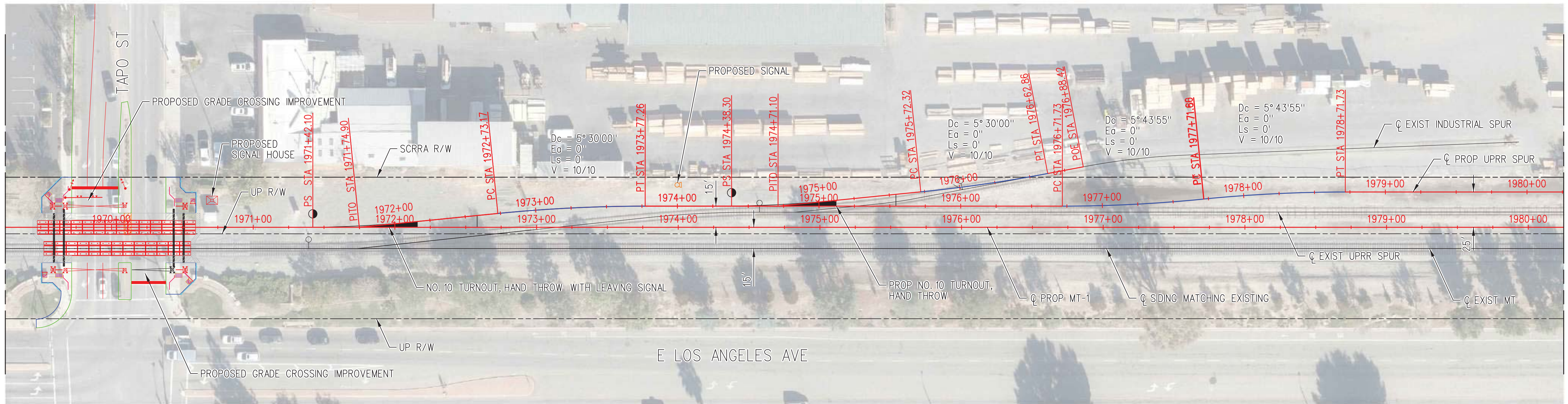
ALTERNATIVE 2
SHEET 6 OF 12

CONTRACT NO. C0000-00
DRAWING NO. T06
REVISION
SHEET NO. 0000 OF 0000
SCALE AS NOTED

TIMETABLE WEST
TO MOORPARK

TIMETABLE EAST
TO LOS ANGELES



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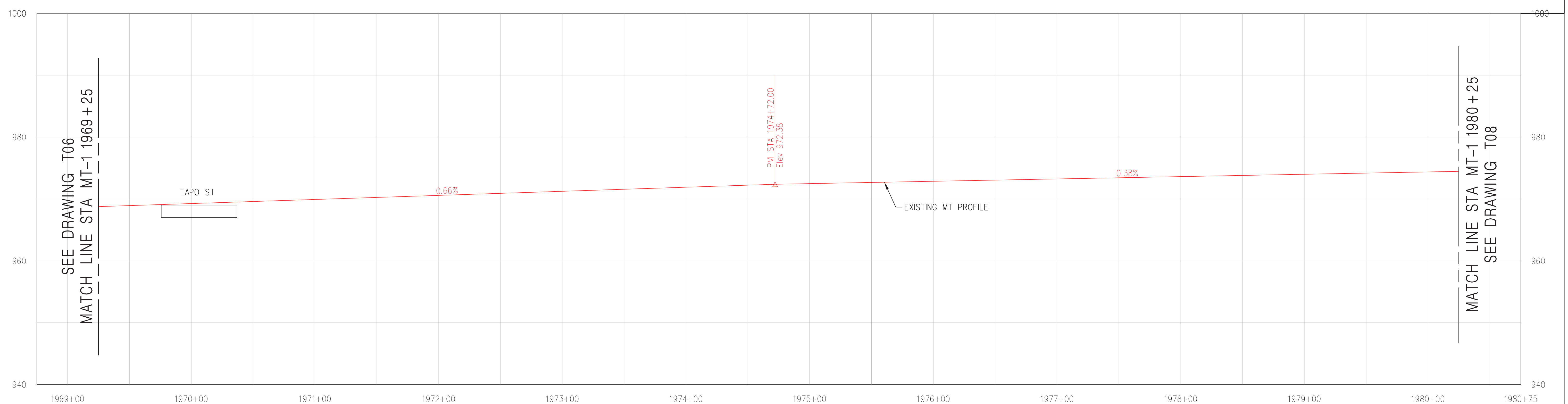
3. PROPOSED GRADE CROSSING IMPROVEMENT TO BE QUIET ZONE READY

100' 0' 100' 200'
GRAPHIC SCALE

LEGEND
- EXIST TRACK
- SCRRA ROW
- UP ROW
- EXIST BRIDGE
- NEW BRIDGE
- EXIST CULVERT
- NEW CULVERT
- NEW TRACK
- SHIFTED TRACK
- NEW CURVE TRACK
- SHIFTED CURVE TRACK
- EXIST GRADE XING PANEL
- NEW GRADE XING PANEL
- NEW RETAINING WALL
- NEW FENCE
- NEW PLATFORM
- NEW SIGNAL



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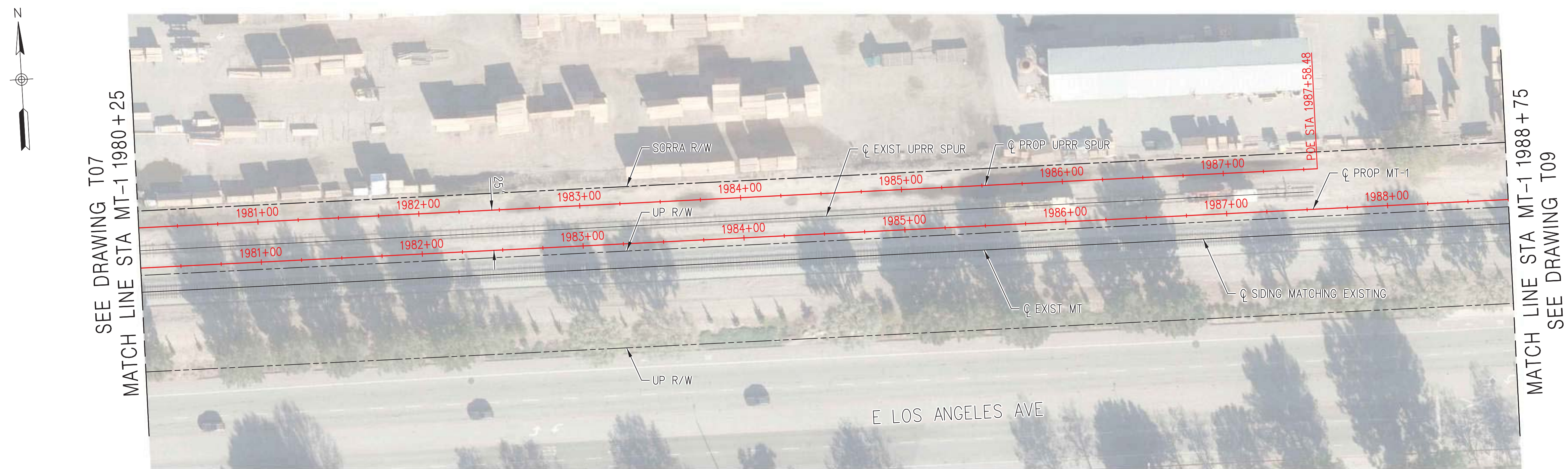
SIMI VALLEY DOUBLE TRACK AND PLATFORM

ALTERNATIVE 2
SHEET 7 OF 12

CONTRACT NO. C0000-00
DRAWING NO. T07
REVISION
SHEET NO. 0000 OF 0000
SCALE AS NOTED



NOTE:
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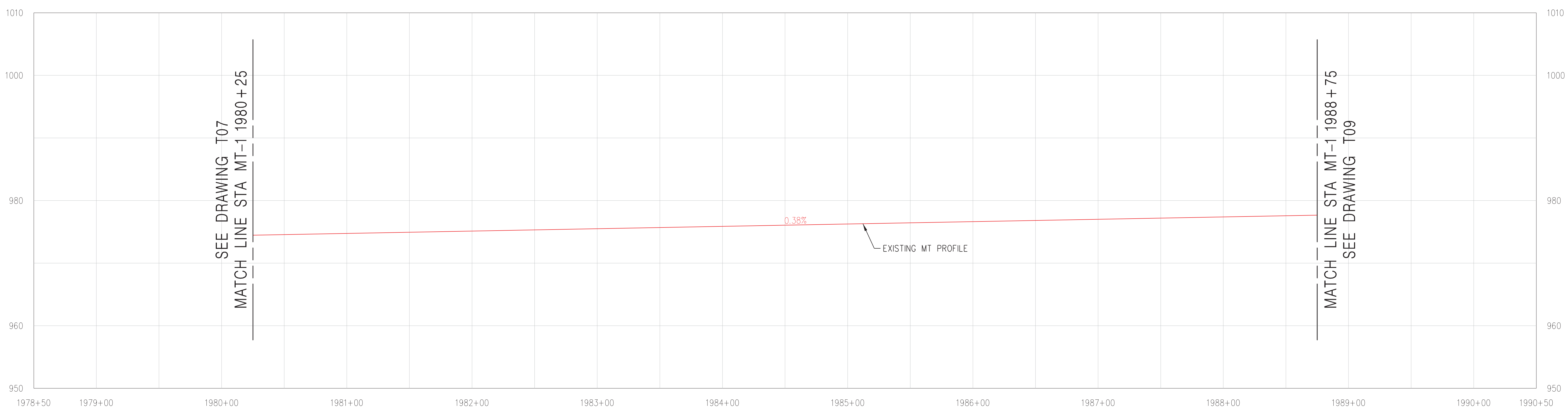
100' 0' 100' 200'
GRAPHIC SCALE

LEGEND

- EXIST TRACK
- SCRRA ROW
- UP ROW
- EXIST BRIDGE
- NEW BRIDGE
- EXIST CULVERT
- NEW CULVERT
- NEW TRACK
- SHIFTED TRACK
- NEW CURVE TRACK
- SHIFTED CURVE TRACK
- EXIST GRADE XING PANEL
- NEW GRADE XING PANEL
- NEW RETAINING WALL
- NEW FENCE
- NEW PLATFORM
- NEW SIGNAL



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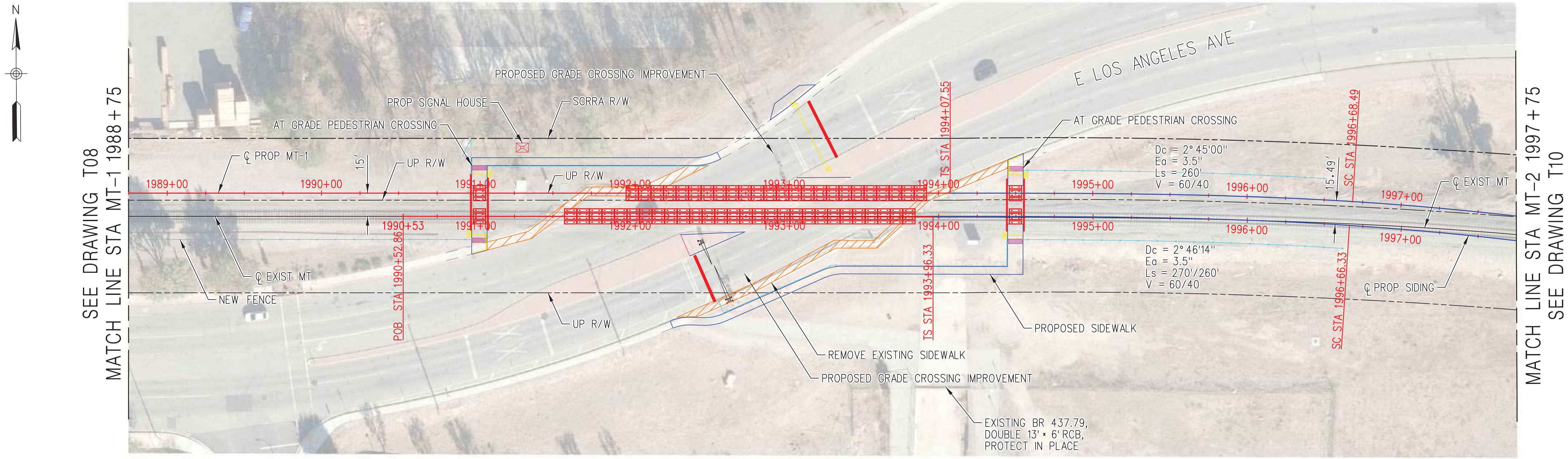
SIMI VALLEY DOUBLE TRACK AND PLATFORM

ALTERNATIVE 2
SHEET 8 OF 12

CONTRACT NO. C0000-00
DRAWING NO. T08
REVISION
SHEET NO. 0000 OF 0000
SCALE AS NOTED

TIMETABLE WEST
TO MOORPARK

TIMETABLE EAST
TO LOS ANGELES



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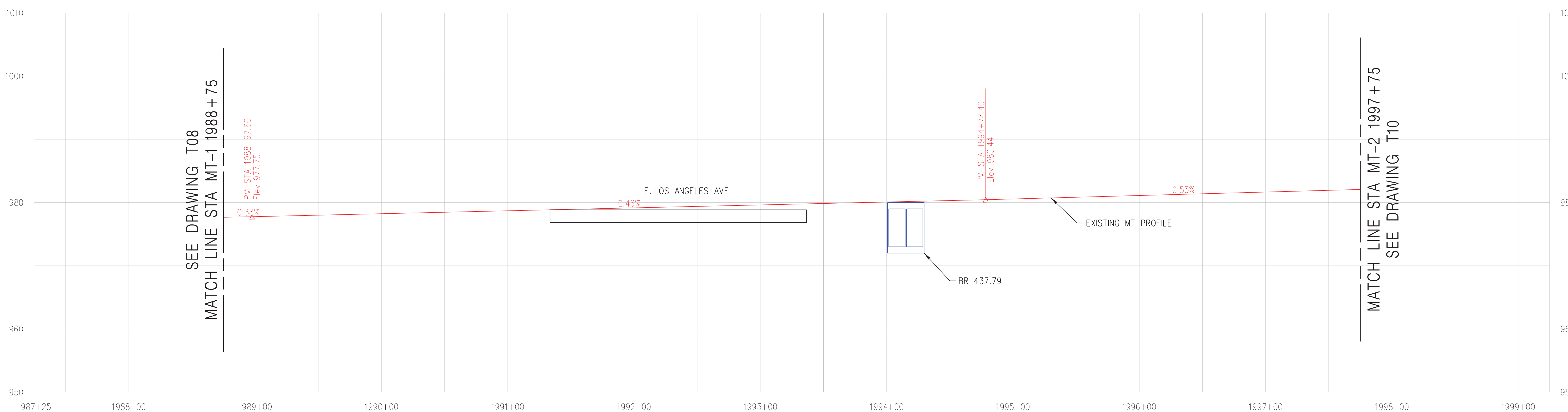
3. PROPOSED GRADE CROSSING IMPROVEMENT TO BE QUIET ZONE READY

100' 0' 100' 200'
GRAPHIC SCALE

LEGEND

NEW TRACK	EXIST TRACK
SHIFTED TRACK	SCRRA ROW
NEW CURVE TRACK	UP ROW
SHIFTED CURVE TRACK	EXIST BRIDGE
	NEW BRIDGE
	EXIST CULVERT
	NEW CULVERT
	EXIST GRADE XING PANEL
	NEW GRADE XING PANEL
	NEW RETAINING WALL
	NEW FENCE
	NEW PLATFORM
	NEW SIGNAL



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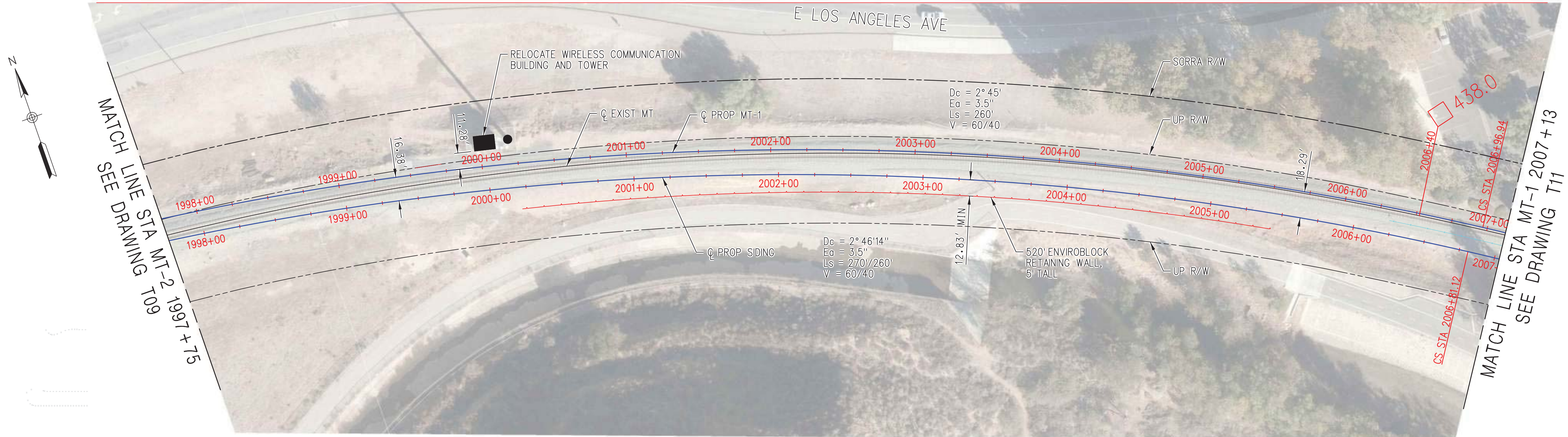
SIMI VALLEY DOUBLE TRACK AND PLATFORM

ALTERNATIVE 2
SHEET 9 OF 12

CONTRACT NO. C0000-00
DRAWING NO. T09
REVISION
SHEET NO. 0000 OF 0000
SCALE AS NOTED



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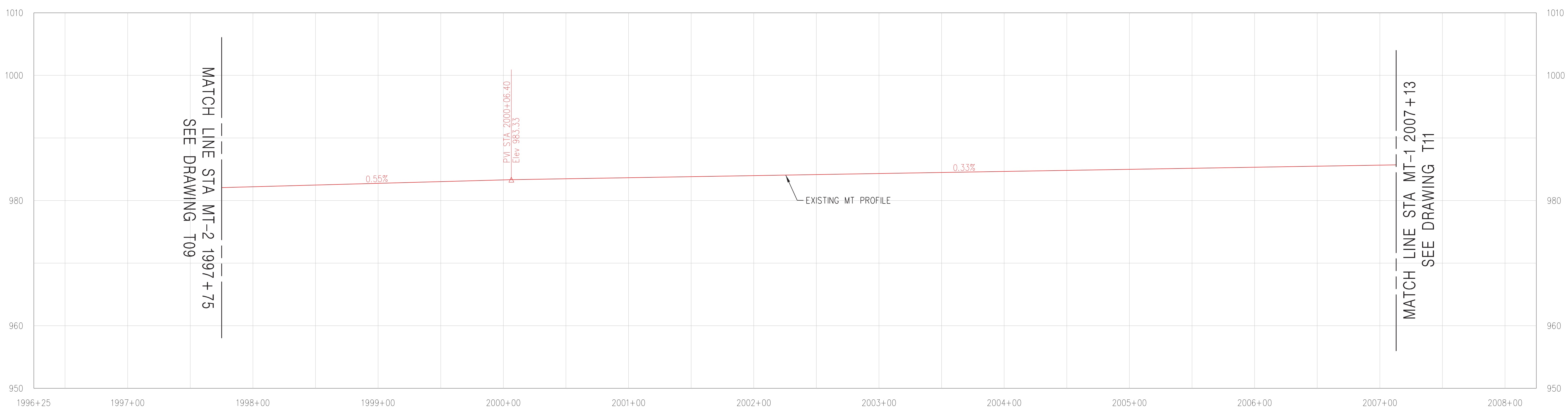
100' 0' 100' 200'
GRAPHIC SCALE

LEGEND

EXIST TRACK	EXIST GRADE XING PANEL
SCRRA ROW	NEW GRADE XING PANEL
UP ROW	NEW RETAINING WALL
EXIST BRIDGE	NEW FENCE
NEW BRIDGE	NEW PLATFORM
EXIST CULVERT	NEW SIGNAL
NEW CULVERT	
NEW TRACK	
SHIFTED TRACK	
NEW CURVE TRACK	
SHIFTED CURVE TRACK	



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SIMI VALLEY DOUBLE TRACK AND PLATFORM

ALTERNATIVE 2
SHEET 10 OF 12

CONTRACT NO. C0000-00
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REVISION
SHEET NO. 0000 OF 0000
SCALE AS NOTED

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TIMETABLE WEST
TO MOORPARK

TIMETABLE EAST
TO LOS ANGELES

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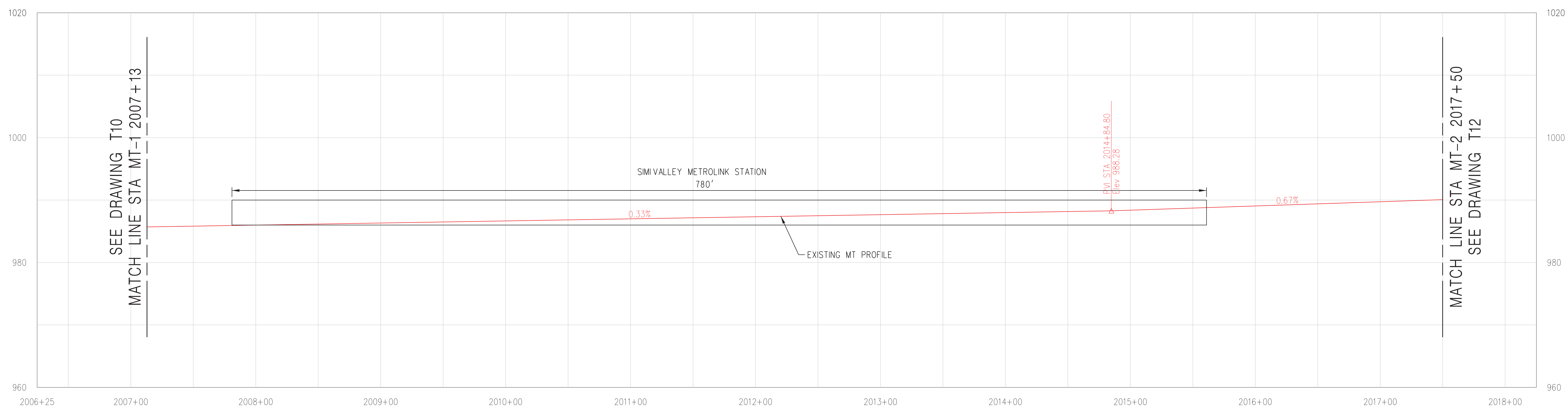
100' 0' 100' 200'
GRAPHIC SCALE

LEGEND

NEW TRACK	EXIST TRACK	EXIST GRADE XING PANEL
SHIFTED TRACK	SCRRA ROW	NEW GRADE XING PANEL
NEW CURVE TRACK	UP ROW	NEW RETAINING WALL
SHIFTED CURVE TRACK	EXIST BRIDGE	NEW FENCE
	NEW BRIDGE	NEW PLATFORM
	EXIST CULVERT	NEW SIGNAL
	NEW CULVERT	



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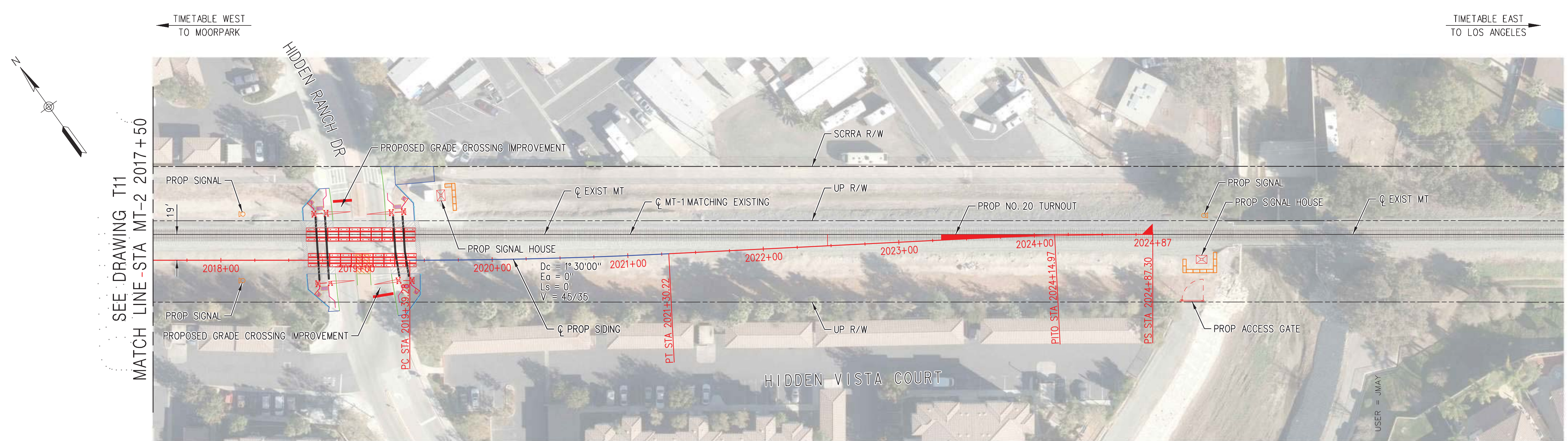
SIMI VALLEY DOUBLE TRACK AND PLATFORM

ALTERNATIVE 2
SHEET 11 OF 12

CONTRACT NO. C0000-00
DRAWING NO. T11
REVISION
SHEET NO. 0000 OF 0000
SCALE AS NOTED



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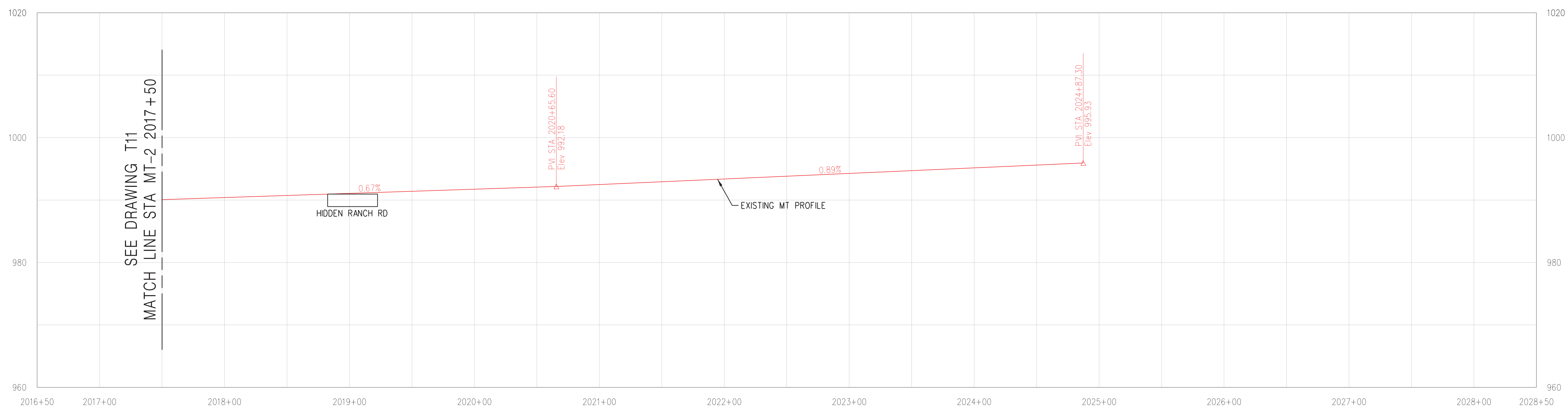
100' 0' 100' 200'
GRAPHIC SCALE

LEGEND
- EXIST TRACK
- SCRRA ROW
- UP ROW
- EXIST BRIDGE
- NEW BRIDGE
- EXIST CULVERT
- NEW CULVERT
- NEW TRACK
- SHIFTED TRACK
- NEW CURVE TRACK
- SHIFTED CURVE TRACK
- EXIST GRADE XING PANEL
- NEW GRADE XING PANEL
- NEW RETAINING WALL
- NEW FENCE
- NEW PLATFORM
- NEW SIGNAL



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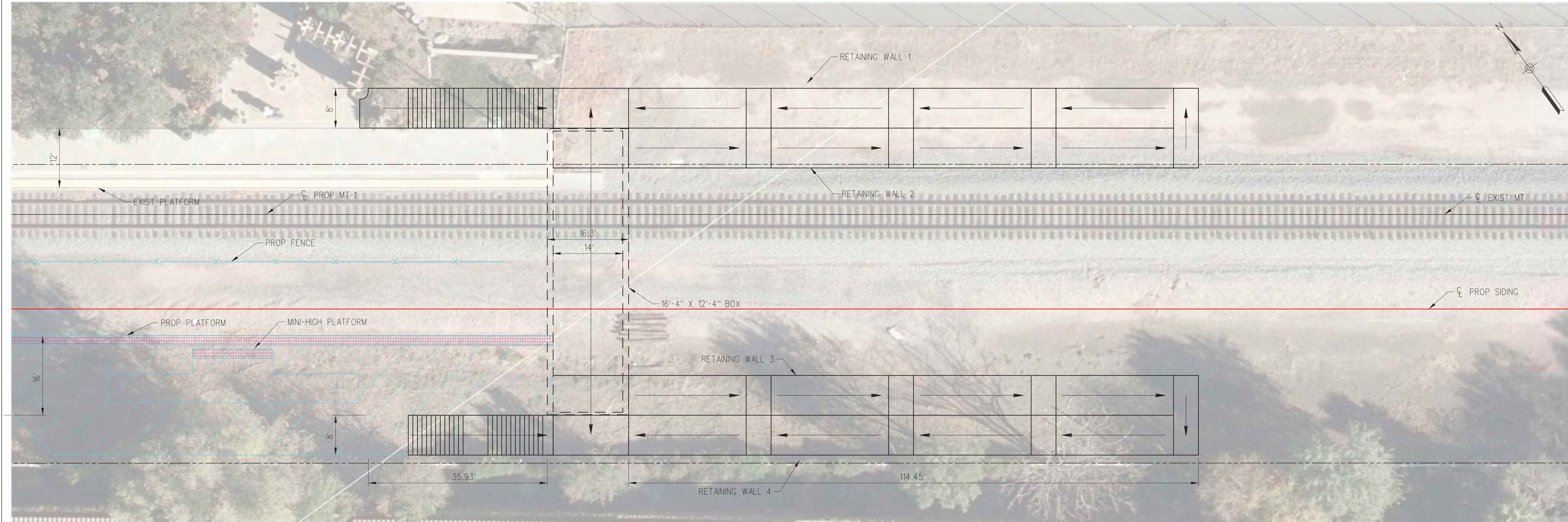
APPROVED:

VENTURA SCORE PHASE I PDR
SIMI VALLEY DOUBLE TRACK AND PLATFORM
ALTERNATIVE 2
SHEET 12 OF 12

CONTRACT NO. C0000-00
DRAWING NO. T12
REVISION
SHEET NO. 0000 OF 0000
SCALE AS NOTED



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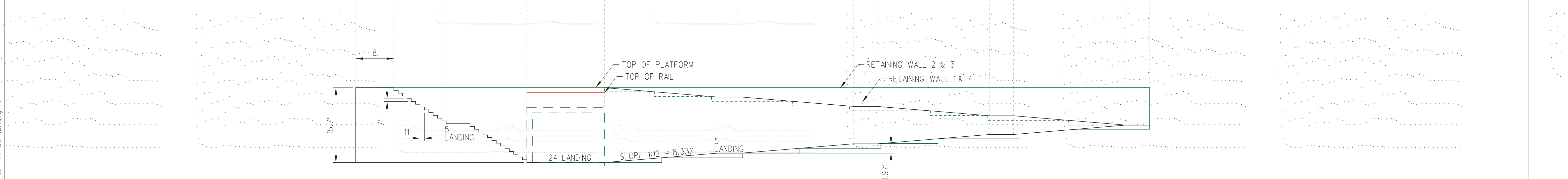

PROFILE

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SIMI VALLEY DOUBLE TRACK AND PLATFORM

PEDESTRIAN UNDPERASS

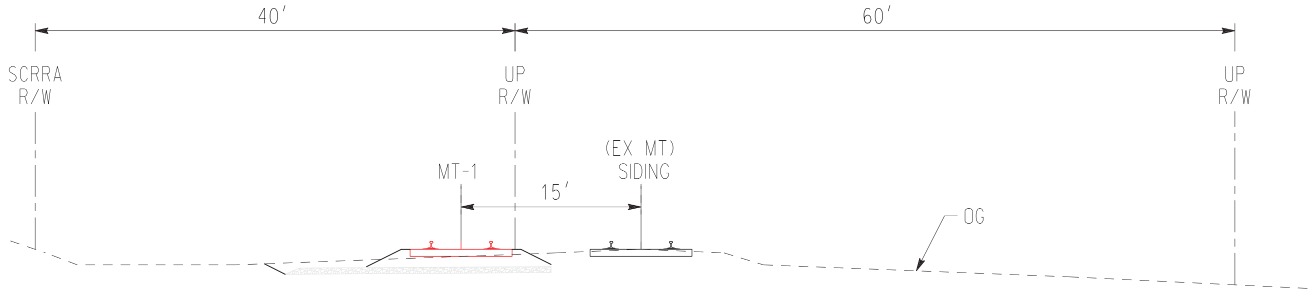
CONTRACT NO. C0000-00
DRAWING NO. T13
REVISION
SHEET NO. 0000 OF 0000
SCALE AS NOTED

EMBANKMENT FILL

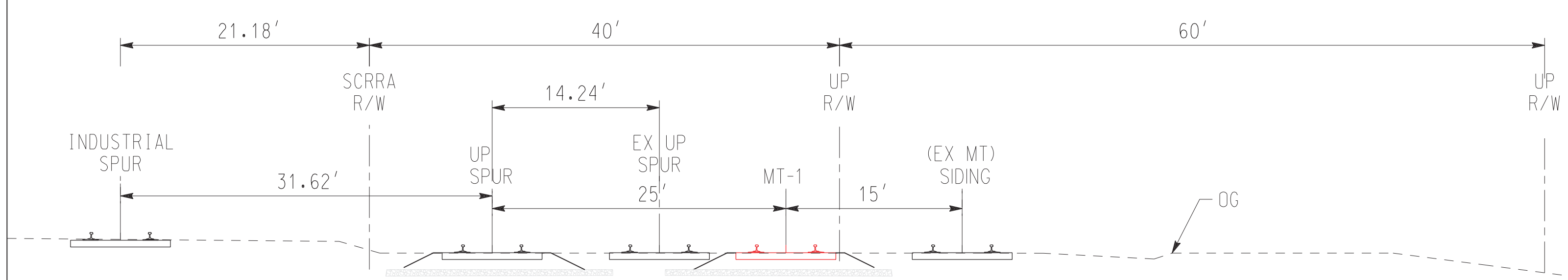
EXISTING SUBGRADE

NEW TRACK

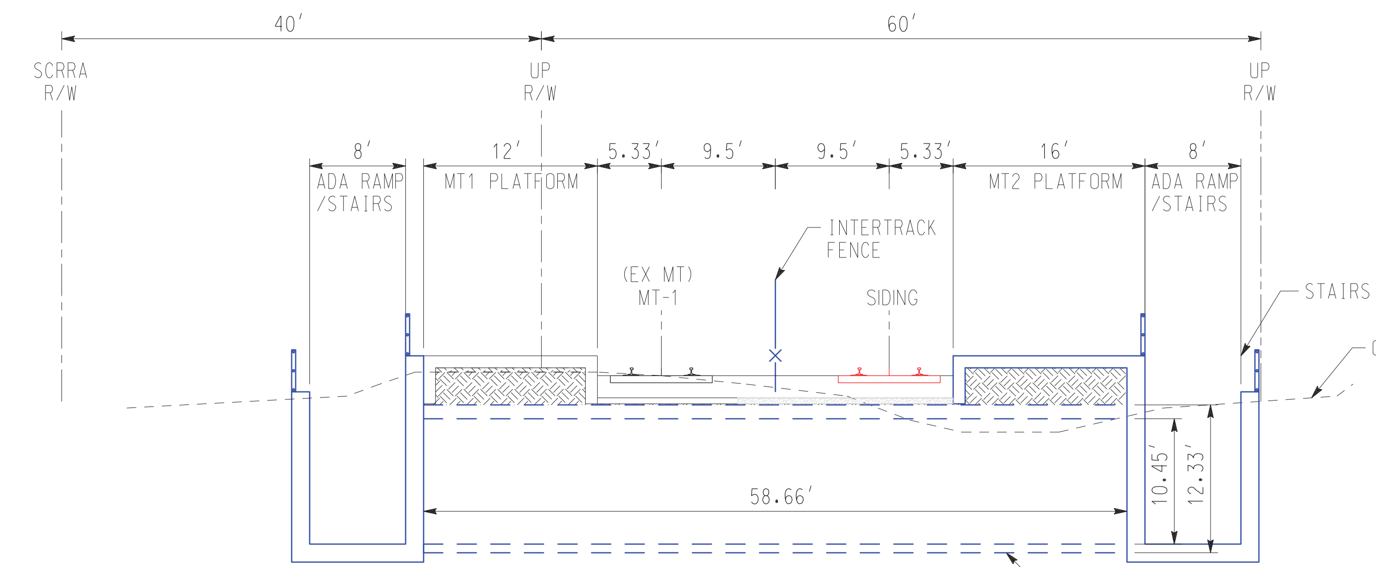
NEW STRUCTURE



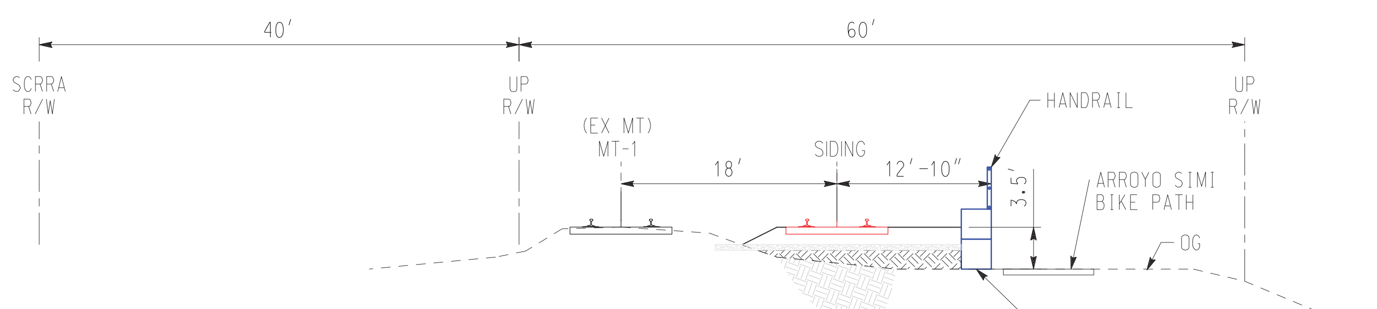
SECTION A
TWO TRACKS - STA 1920+00



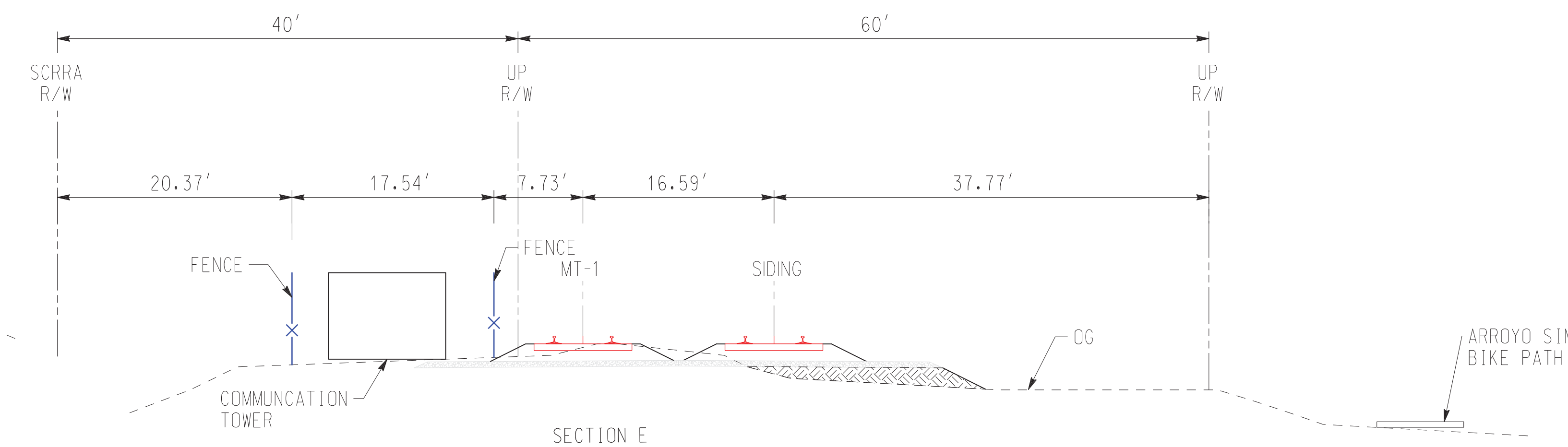
SECTION B
FOUR TRACKS - STA 1979+00



SECTION D
SIMI VALLEY STATION - STA 2010+00



SECTION C
TWO TRACKS WITH RETAINING WALL- STA 2004+00



SECTION E
TWO TRACKS WITH COMMUNCATION TOWER- STA 2000+00

REV. | DATE | BY | SUB. | APP.

DESIGNED BY D.ESIGNER
DRAWN BY D.RAWN
CHECKED BY C.HECKER
APPROVED BY I.NCHARGE
DATE XX-XX-XXXX

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VENTURA SCORE PHASE I PDR
SIMI VALLEY DOUBLE TRACK AND PLATFORM

ALTERNATIVE 2
TYPICAL SECTIONS

CONTRACT NO. C0000-00
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REVISION
SHEET NO. OF 0000
SCALE