Gilman Springs Road Phase 6 Safety Project

From 8,920 Feet S/O Alessandro Boulevard To 5,340 Feet S/O Bridge Street

Eastern Moreno Valley and Gilman Hot Springs Areas

Project No. C2-0161

Federal Aid No. HSIPL-5956 (263)

Geotechnical Report
Gilman Springs Road Improvements
Dated October 6, 2020

Notice regarding this Report:

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A Report Prepared for:

NCM Engineering Corporation
4740 Green River Road, Suite 218
Corona, CA 92880

GEOTECHNICAL REPORT
GILMAN SPRINGS ROAD IMPROVEMENTS
RIVERSIDE COUNTY, CALIFORNIA

Project No. 2018-019

by

Britton Howay, EIT
Staff Engineer II

REGISTERED PROFESSIONAL ENGINEER
SOMADEVAN NIRANJANAN
No. GE2819
GEOTECHNICAL
STATE OF CALIFORNIA

Somadevan Niranjanan
Geotechnical Engineer 2819

Diaz•Yourman & Associates
1616 East 17th Street
Santa Ana, CA 92705-8509
(714) 245-2920

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1 INTRODUCTION

This report presents the results of the preliminary geotechnical engineering services performed by Diaz•Yourman & Associates (DYA) for the proposed shoulder widening of Gilman Springs Road, which is located in the rural unincorporated Moreno Valley area of Riverside County. Our services are to support the Preliminary Engineering and Environmental Documentation Phase of the Project. The proposed shoulder widening is planned along Gilman Springs Road for an approximately 4.4-mile-long segment between 8,900 feet south of Alessandro Boulevard and 5,100 feet south of Bridge Street. NCM Engineering Corporation authorized this work on February 26, 2019.

The Project alignment is shown on the Vicinity Map, Figure 1. The proposed Project will consist of widening the existing roadway to provide one 4-foot wide double yellow striped center median with impact resistant channelizers and rumble strips, one 12-foot wide lane in each direction, 5-foot-wide paved shoulders with rumble strips, and a 6-foot-wide graded shoulder. The widening will also include one, approximately 6,900-foot long, passing lane in the westbound direction. Additionally, the Project will replace the existing reinforced concrete box culvert near Bridge Street with a single-span concrete slab bridge to serve as a wildlife crossing. Also, the Project proposes three retaining walls, approximately 10 to 16 feet high and approximately 100 to 320 feet long. The proposed improvements will also require site grading, extension of existing drainage facilities, and relocating utilities. We understand the roadway was realigned and new pavement was constructed about 6 years ago. The approximate layout of the proposed Project is shown on the Site Plan in Appendix A. The proposed roadway grades will be near existing grade.

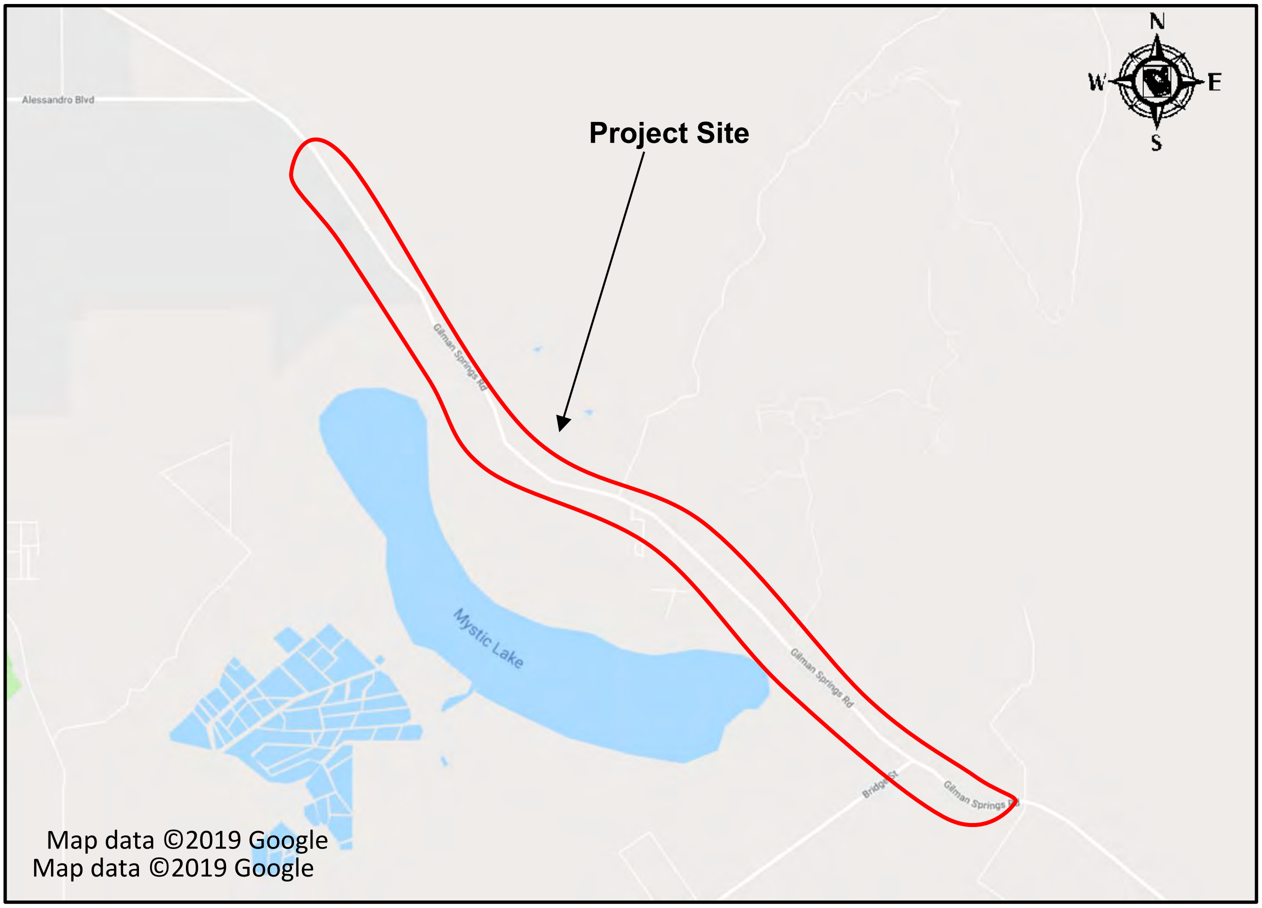

Figure 1 - VICINITY MAP

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

- Reviewing existing data.
- Obtaining an encroachment permit from County of Riverside.
- Marking soil boring locations at the site and notifying underground service alert (USA) for underground utility clearance.
- Conducting a limited geotechnical field exploration.
- Performing a limited geological site reconnaissance.
- Performing geotechnical laboratory tests on selected soil samples.
- Performing engineering analyses to develop preliminary conclusions and recommendations regarding the following:
    - Subsurface conditions
    - Geologic and seismic hazards
    - Site preparation and grading
    - Foundation type and allowable bearing capacity
    - Foundation settlements
    - Resistance to lateral loads
    - Lateral earth pressures
    - New pavement structural sections
    - Soil corrosion potential
- Preparing this preliminary geotechnical report.

Evaluation of the existing pavements surface conditions and or any pavement rehabilitation recommendations are not part of our scope of services. We understand that the County is performing such investigation. Fault rupture analyses, site-specific liquefaction analysis, and analyses related to potential landslide at the proposed retaining walls site are also excluded from our scope of services.

2 DATA REVIEW, FIELD EXPLORATION, AND LABORATORY TESTING

Geotechnical data from the Project vicinity presented in previous reports were reviewed to supplement site data collected during this exploration. A list of the documents reviewed is presented in the bibliography (Section 9).

The field exploration, conducted on April 25 and 26, 2019, and May 3, 2019, consisted of drilling a total of 14 borings and collecting surface bulk samples at 8 locations as summarized below:

- Eleven borings to an approximate depth of 6.5 to 12 feet below ground surface (bgs) for road widening.
- Three borings to an approximate depth of 31.5 to 41.5 feet bgs for the three proposed retaining walls.
- Eight bulk samples from the surface to a few feet bgs exclusively for corrosion tests for the proposed culvert widening.

The borings and surface bulk sampling locations are shown on the site plan in Appendix A. DYA initially recommended that more borings be performed; however, the number of borings required for this phase of the Project was requested by the County of Riverside. The borings and surface bulk sampling locations were carefully chosen with the consultation of NCM Engineering to avoid any underground utility conflicts and environmentally sensitive areas and to provide areal coverage of the Project site for the preliminary engineering study. Details of the field exploration, including sampling procedures and boring logs, are presented in Appendix B.

Soil samples collected from the borings at the site were re-examined in our laboratory to substantiate field classifications as part of our quality control procedures. Selected soil samples were tested for moisture content, dry density, grain-size distribution, percent passing the No. 200 sieve, Atterberg limits, expansion index, shear strength, compaction characteristics, R-Value, sand equivalent, and corrosion potential. The soil samples tested are identified on the boring logs. Laboratory test data are summarized on the boring logs in Appendix B and presented on individual test reports in Appendix C.

3 SITE CONDITIONS

3.1 GEOLOGY

The Project alignment lies within the Peninsular Ranges Geomorphic Province of Southern California. The Peninsular Ranges are a series of northwest-trending ranges that extend into lower California and the Baja California Peninsula (CGS 2002). The alignment crosses a number of mapped units including Quaternary aged axial-valley (Qya), alluvial fan (Qyf, Qofu), and old landslide deposits (Qols). These deposits are typically described as consisting of sand and gravel. Quaternary aged deposits can be varying thicknesses. Tertiary aged Arkosic sandstone overlying lithic conglomerate (Tmea) also underlies the Project alignment as well as Cretaceous aged Tonalite. The Tonalite bedrock is noted as being highly fractured and weathered (Morton, 2001, Morton 2015).

3.2 SURFACE CONDITIONS

The Project site is located in a mainly agricultural area with sparse single-family residential homes. Gilman Springs Road serves as the travel way for commuters and agricultural transportation as it runs in a northwest to southeast direction between State Route (SR)-60 and Highway 79. Mystic Lake is located on the west side of the Project alignment and experiences fluctuating levels of groundwater, appearing dry in drought seasons and full during heavy rainy seasons, such as at the time of our field exploration. The existing roadway within the Project limit consisted of mainly one lane in each direction except for an occasional left- and right-turn pocket lanes. There are numerous culverts and overhead powerlines along the Project alignment. Gilman Springs Road is currently paved with asphalt concrete (AC). At the time of the initial site reconnaissance, the existing AC pavement surface conditions were in very poor to fair condition and exhibited distresses in the form of cracking, deformation, and deterioration. Types of pavement distresses observed along the Project alignment were longitudinal & transverse cracking, alligator cracking, block cracking, depressions, potholes, and patched areas. Some longitudinal cracks were observed to be as long as 30 feet, and some transverse cracks were as wide as the roadway width. On March 26, 2019, there was a reported sinkhole near Bridge Street, and during our site reconnaissance, it was observed that the sink hole was patched. Since, the time of our initial site reconnaissance, the roadway within the Project limits has been resurfaced by way of cold-planing and placing new asphalt concrete; and delineators were put in place along the median. Existing ground surface elevation along the approximately 4.4-mile-long Project alignment ranged from 1,432 to 1,549 feet above mean sea level (NAVD88).

The pavement structural sections encountered during DYA's field exploration are summarized in Table 1.

Table 1 - EXISTING PAVEMENT STRUCTURAL SECTIONS

BORING	ASPHALT CONCRETE (inches)	AGGREGATE BASE (inches)
DYB19-09	10	3
DYB19-10	7	8
DYB19-11	4	7

Note(s):
- Pavement thicknesses reported should be considered approximate.
- See Site Plan in Appendix A for boring locations. See Appendix B for boring logs.

3.3 SUBSURFACE CONDITIONS

3.3.1 Roadway

The subsurface conditions along the roadway were evaluated using limited 14 borings performed for the project. In general, in the upper 5 feet, the subsurface soils consisted of loose to medium-dense silty sand and stiff sandy lean clay. Because the subsurface soils conditions vary along the Project Alignment, if a location-specific subsurface condition is required, please review the corresponding boring log provided in Appendix B.

A summary of the test results for in situ moisture content, dry unit weight, optimum moisture content, and maximum dry unit weight are provided in Table 2. The relative compaction values presented can be considered an indication of the subsurface soil conditions but may not provide an actual representation because of sample disturbance during sampling and transportation.

Based on the Atterberg tests (plasticity index [PI] ranged from 14 to 26, and liquid limit ranged from 30 to 39) performed on selected fine-grained soils, the on-site, fine-grained soil can be classified to have low to medium expansion potential based on a PI relationship interpretation (USBR, 1998, Earth Manual Third Edition). We performed two expansion index (EI) tests on the upper soil layer (upper 5 feet) in Borings DYB19-03 and DYB19-13, and the EI test results were 0 and 43, respectively. Based on the EI tests, the expansion potential is low. Therefore, we conclude that the fine-grained soils at the site have a low to medium potential for expansion.

Expansive soils will undergo changes in volume with changes in moisture content (expand when saturated and shrink when dried), which can result in lifting and cracking of flatwork.

Table 2 - SUBSURFACE SOIL CHARACTERISTICS

BORING ID	DEPTH (feet)	SOIL TYPE	IN SITU MOISTURE CONTENT (%)	OPTIMUM MOISTURE CONTENT[1] (%)	IN SITU DRY UNIT WEIGHT (pcf)	MAXIMUM DRY UNIT WEIGHT[1] (pcf)	RELATIVE COMPACTION (%)
DYB19-01	5	SM	4.9	--	111.0	--	--
DYB19-02	1.5	CL	8.3	8.4	111.1	133.6	83
DYB19-03	5	SM	4.9	7.6	111.9	133.4	84
DYB19-05	3	CL	14.3	--	110.1	--	--
DYB19-07	1.5	SM	4.7	7.5	99.2	133.7	74
DYB19-08	2.5	SP-SM	3.4	--	110.1	--	--
DYB19-09	5	SM	7.1	--	99.4	--	--
DYB19-10	2.5	SM	8.7	--	109.9	--	--
DYB19-11	2.5	SM	2.5	--	115.5	--	--
DYB19-12	1.5	SM	6.9	--	96.7	--	--
DYB19-13	0-5	SM & CL	--	10.4	--	127.8	--

Note(s):

1. Based on the bulk samples from the upper 5 feet.

- Soil classification based on ASTM Soil Classification System (ASTM D2487 and 2488).
- In situ moisture contents and in situ dry weights, along with optimum moisture contents and maximum dry densities were obtained from laboratory testing. The relative compaction values presented above may be an indication of the subsurface soil conditions but may not represent the actual representation due to sample disturbance during sampling and transportation.
- pcf = pounds per cubic foot.

3.3.2 Retaining Wall

Based on review of Borings DYB19-09 through DYB19-11, in general, loose to medium-dense (approximately upper 15) to very dense (approximately below 15 feet bgs) silty sands were encountered to a depth of approximately 40 feet near the proposed retaining wall locations.

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Table 3 - IDEALIZED SUBSURFACE DESIGN PROFILE (RETAINING WALLS)

WALL ID[1]	SOIL LAYER[2]	ELEVATION (feet)	DEPTH (feet)	TOTAL UNIT WEIGHT (pcf)	FRICTION ANGLE (degrees)	COHESION (psf)
1	Medium dense, (SM)	1,510 – 1,493	0 – 17	115	30	100
	Medium dense to Very dense, (SM)	1,493 – 1,468	17 – 42	120	33	100
2	Loose to medium dense, (SM)	1,505 – 1,498	0 - 7	115	30	100
	Medium dense to very dense, (SM)	1,498 – 1473	7 – 32	120	33	100
3	Loose to medium dense, (SM)	1,494 – 1,480	0 - 14	115	30	100
	Medium dense to very dense, (SM)	1,480 – 1,455	14 – 39	120	33	100

Note(s):

1. Approximate Stations for the proposed retaining walls:
    - Retaining Wall 1: 343+56 to 345+00
    - Retaining Wall 2: 346+50 to 347+50
    - Retaining Wall 3: 348+90 to 351+60
2. Predominant soil type.
    - SM = Silty Sand; pcf = pounds per cubic foot.

Groundwater was not encountered to the depths of exploration during drilling operations. California Geological Survey (CGS) has not yet prepared historically highest groundwater maps for the El Casco and Lakeview 7.5-minute quadrangles that encompass the Project alignment. Mystic Lake is located on the west side and adjacent to the Project alignment and experiences fluctuating levels of groundwater, appearing dry in seasons of drought and full during heavy rainy seasons, such as at the time of this Project's field exploration.

GeoTrackerGAMA website was checked for any possible groundwater information. In the GeoTrackerGAMA (2019) database, several groundwater monitoring wells data were found. Based on the review of the well records, the depth to groundwater could be as deep as 100 feet.

Table 4 provides a summary of the selected groundwater well information based on review of GeoTrackerGAMA (2019).

Table 4 - SUMMARY OF GROUNDWATER LEVELS

WELL	WELL ID[1]	OBSERVATION DURATION	DEPTH TO GROUNDWATER (feet, bgs)	APPROXIMATE GROUND WATER ELEVATION (feet, MSL)
1	338615N1170375W001	11/2011 through 09/2015	182.7 to 192.7	1,302 to1,291
2	338646N1170600W001	10/2011 through 10/2015	192.6 to 194.3	1,233 to 1,231
3	339036N1171216W001	10/2011 through 09/2015	143.7 to 162.6	1,210 to 1,197
4	338765N1170922W001	10/2011 through 10/2015	113.5 to 128.8	1,314 to 1,299
5	338691N1170839W001	10/2011 through 10/2015	217.2 to 230.8	1,351 to 1,332

Note(s):
1. Based on available data through GeoTrackerGAMA, (2019).

Based on available groundwater information, we judged the design groundwater level at the proposed retaining wall site to be 50 feet bgs or deeper for design purposes.

4 CONCLUSIONS AND RECOMMENDATIONS

Based on geotechnical considerations, the site is suitable for the proposed Project. The following geotechnical considerations need to be carefully considered:

- Fault Rupture – The majority of the site is located within an Alquist-Priolo (AP) zone (CGS 1995). Therefore, the potential for fault rupture is very high. The consideration for the AP zone is important for occupancy buildings, bridges, and major/important pipelines. A Project like this that involves pavements and retaining walls are typically not given much consideration for fault rupture hazard. The mitigation cost for fault rupture is significant when compared to the cost of the project. Typically, it is common to repair after a rupture event. However, we recommend that this be discussed with the County of Riverside during the project design meeting.
- Liquefaction – Liquefaction is a phenomenon whereby saturated sediments temporarily lose their shear strength and collapse during seismic shaking. This condition is caused by cyclic loading during earthquake shaking that generates high porewater pressures within the sediments. The soil types most susceptible to liquefaction is loose, cohesionless, and granular soils that are below the water table. Groundwater was not encountered during our explorations to a depth of 41.5 feet. In the vicinity of the Project alignment, numerous groundwater wells exist that were installed for environmental investigations. Based on review of those wells, we can conclude that the site groundwater will be deeper than 50 feet. Therefore, liquefaction-induced settlement and consequences are considered remote. However, according to the Reche Canyon/Badlands Area Plan (2017), the Project alignment is in a low to moderate liquefaction zone with susceptible sediments in deep groundwater. Based on the above considerations and planned project improvements, further liquefaction analyses are not warranted.
- Stability of steeper slopes – The proposed widening and the ROW restrictions due to environmentally sensitive areas adjacent to Project site will cause cut slopes in some areas to be steeper than 2H:1V. An engineering geologic study presented in a Technical Memorandum in Appendix D of this report has determined that cut slope gradients between 2H:1V and 1H:1V at the areas specified should result in grossly stable excavations (Technical Memorandum, 2019).

- Soil Types – The project site consisted of coarse and fine-grained soils in the upper five feet along the roadway alignment. Moisture content varies significantly from the optimum moisture content. Therefore, proper moisture conditioning will be key to successful Project grading.
- Retaining Wall Foundations – We recommend that the proposed retaining walls can be supported on shallow foundations. The retaining wall construction will require either shoring or slope back cut. Currently, we are unaware of the construction methodology.

4.1 GROUND RUPTURE

Portions of the Project alignment, including the proposed retaining walls, are located within the Alquist-Priolo Earthquake Fault Zone ([APEFZ]; CGS, 1995). See Figure 2 for AP Zone Hazard Map. Therefore, we consider the possibility of surface rupture at the Proposed retaining wall locations to be high. Our understanding is that the Project may not require a detailed fault rupture investigation because no inhabited structures are planned. An investigation on the fault rupture is not part of the scope of our service.

4.2 SEISMIC/GEOLOGIC HAZARDS

The site, like most of Southern California, will be subject to strong ground shaking during major earthquakes. See Table 5 for the closest faults to the Project site based on the Caltrans fault database (Caltrans, 2012).

Table 5 - MAJOR FAULT CHARACTERIZATION IN THE PROJECT VICINITY

FAULT[1]	FID[2]	SITE-TO-FAULT DISTANCE (km) R_X[3]	SITE-TO-FAULT DISTANCE (km) R_{RUP}[4]	TYPE[5]	M_{MAX}[6]	DIP AND DIRECTION[7] (degree/direction)
San Jacinto (San Jacinto Valley)	356	0.05	0.05	SS	7.7	90 Deg/V
San Jacinto (Anza)	362	3.20	3.20	SS	7.7	90 Deg/V
San Jacinto (San Bernardino Valley)	310	1.15	12.96	SS	7.7	90 Deg/V

Note(s):

1. Based on Caltrans Accelerated Response Spectrum (ARS) Online Tool (Caltrans, 2019).
2. FID = Fault Identification Number.
3. R_X distance is defined as the closest distance to the fault trace or surface projection of the top of the rupture plane.
4. R_{RUP} is defined as the closest distance from the fault rupture plane. The distance measurements are approximate.
5. SS = Strike Slip.
6. M_{MAX} = Maximum earthquake magnitude.
7. V = vertical.

- The site location is 33.889647 N and 117.071229 W.

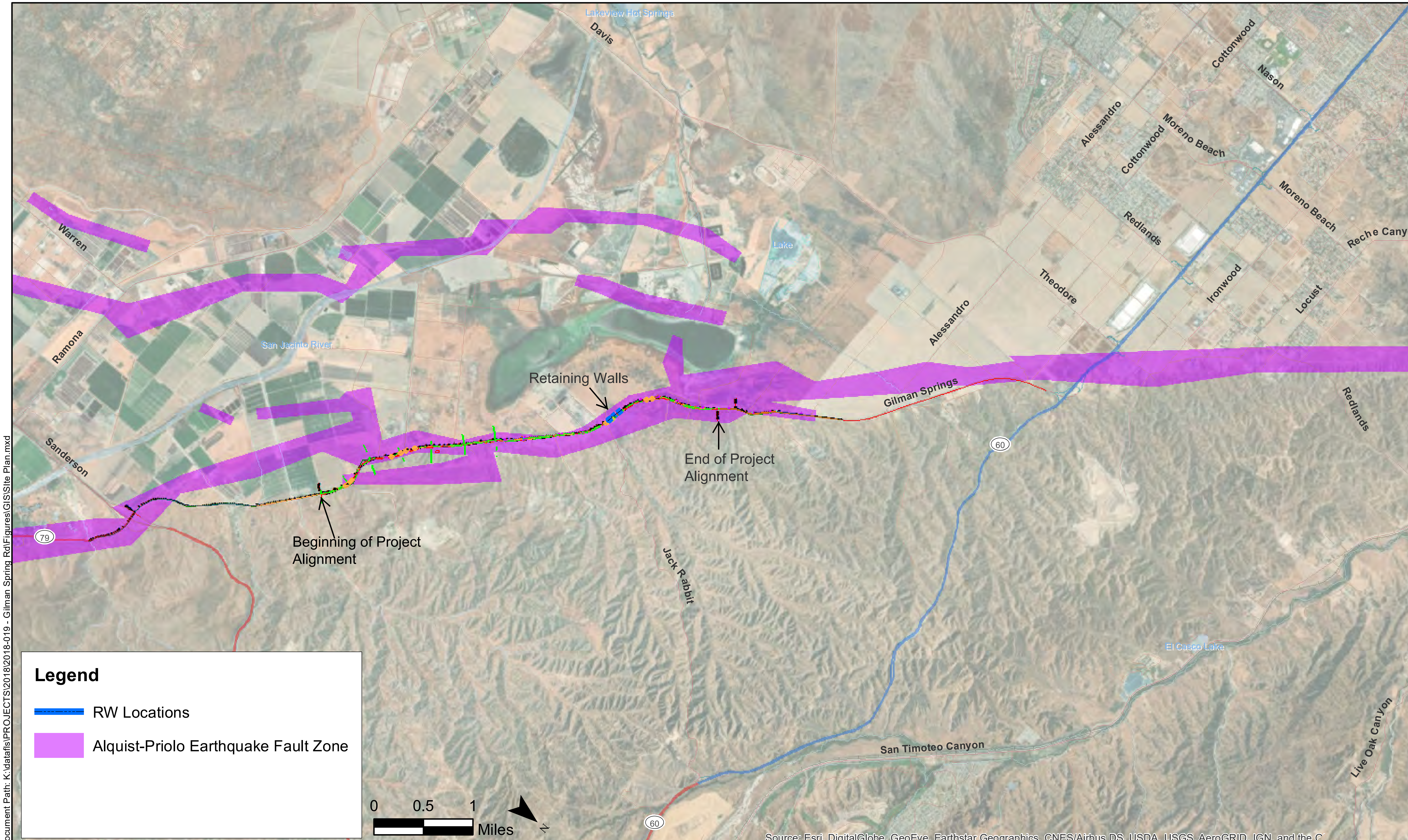

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Figure 2 - AP Zone Hazard Map

A design 5% damped horizontal Acceleration Response Spectrum (ARS) based on Caltrans seismic design procedures (Caltrans, 2013) was developed for the proposed retaining walls site. The Caltrans seismic design procedures consider both deterministic and probabilistic (975-year-return period) approaches and enveloping the spectra developed by using both methodologies. Based on these Caltrans seismic design procedures, the horizontal peak ground acceleration (PGA) for the proposed retaining wall locations was calculated to be approximately 0.94g. The recommended design 5% damped horizontal ARS is summarized in Table 6.

Table 6 - DESIGN HORIZONTAL ACCELERATION RESPONSE SPECTRUM

PERIOD (seconds)	HORIZONTAL 5% DAMPED SPECTRAL ACCELERATION (g)				
	Deterministic			Probabilistic	Design
	San Jacinto (San Jacinto Valley)	San Jacinto (Anza)	San Jacinto (San Bernardino Valley)		
0.01	0.540	0.468	0.293	0.941	0.941
0.05	0.639	0.558	0.354	1.348	1.348
0.1	0.793	0.715	0.496	1.574	1.574
0.15	0.923	0.836	0.595	1.774	1.774
0.2	1.025	0.920	0.636	1.930	1.930
0.25	1.081	0.958	0.640	1.990	1.990
0.3	1.108	0.971	0.632	2.040	2.040
0.4	1.125	0.965	0.595	2.004	2.004
0.5	1.133	0.956	0.565	1.977	1.977
0.6	1.132	0.942	0.542	1.954	1.954
0.7	1.133	0.932	0.525	1.944	1.944
0.85	1.114	0.908	0.501	1.859	1.859
1.0	1.086	0.882	0.480	1.764	1.764
1.2	0.975	0.789	0.425	1.537	1.537
1.5	0.840	0.678	0.360	1.298	1.298
2.0	0.659	0.530	0.277	1.043	1.043
3.0	0.433	0.347	0.179	0.695	0.695
4.0	0.313	0.251	0.129	0.502	0.502
5.0	0.243	0.196	0.101	0.410	0.410

Note(s):

- Design ARS is the envelope of deterministic and probabilistic spectra.

According to the Reche Canyon/Badlands Area Plan (2017), the Project alignment is in a low to moderate liquefaction zone with susceptible sediments. However, we judge that the subsurface soils are not subject to liquefaction based on assumed design groundwater level (approximately 50 feet bgs or deeper) and soil types and consistency.

4.3 EARTHWORK

Earth work operation is anticipated for the proposed shoulder widening and construction of the retaining walls. We recommend that Caltrans standard specifications (Caltrans, 2018a) be adopted for this Project.

4.3.1 Site Preparation and Grading

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

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

Excavations are anticipated for shoulder widening and preparation for retaining wall foundations. The bottom of the excavation should be:

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

The bottom of the excavations should be proof rolled to check for any loose or soft soils prior to placing fill. The bottom of the excavation should be firm, hard, and unyielding. When the subgrade soils at the bottom of the excavation preclude compaction, the subgrade soil should be over excavated to a sufficient depth to achieve a firm and unyielding surface at the planned bottom of over excavation or the base of the fill. There are a couple of options to help reduce the depth of over excavation to competent subgrade.

Using geotextiles with geogrids can reduce the depth of over excavation. The nonwoven geotextile should satisfy the requirements of Standard Specifications for Public Works Construction (Greenbook) Table 213-2.2 (A) nonwoven Type 90N.

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

The geotextile should be installed as follows:

- Place the geotextile in position and manually roll it out over the subgrade.
- Adjacent rolls of geotextiles should be overlapped by at least 2 feet in the direction that the fill would be spread.
- The geotextile corners may be held down with shovelfulls of fill, pins, sandbags, etc.

Place a geogrid as described below

The geogrids should satisfy the requirements of Standard Specifications for Public Works Construction ([Greenbook]; Building News, 2018) Table 213.5.2 (D) Biaxial S1.

The geogrid should be installed by:

- Placing the geogrid on compacted fill that has been smoothed to remove surface obstructions.
- Nailing one end of the geogrid with 6-inch-long "U" staples and/or other approved fasteners to the end edges of the geogrid roll.
- Unrolling the geogrid without dragging.
- Pulling the geogrid taut to remove any slack.
- Placing fill from the fastened geogrid side to the unfastened geogrid side.
- Continually tensioning the geogrid by hand.
- Pinning the remaining end edges of the taut geogrid as described above.
- Overlapping the geogrid by at least 12 inches.
- Overlapping the geogrid in the direction of fill placement.
- Pushing a new layer of fill on top of the geogrid without creating waves in the geogrid and without the construction equipment contacting the geogrid.
- Compacting the new layer of fill as outlined above.
- Repeating the above steps as necessary for the remaining layers of geogrid.

Another option to improve the subgrade soils and thus reduce the depth of over excavation, would be to treat the subgrade soils to a minimum depth of 12 to 18 inches with approximately 3 to 7 percent cement. The recommended cement content range is not based on laboratory testing, but it is based on our past experience with similar soils. According to Caltrans Highway Design Manual Chapter 660 (Topic 664, Subgrade Enhancement) and within Section 664.3 Chemical Stabilization (Caltrans, 2017), low quality in situ subgrade soils can be improved from a Type III to Type II or Type I by chemical stabilization to a minimum depth of 0.65 feet using either a lime or cement type agent. To ensure long-term stability of the subgrade during the pavement design life, the stabilized soil should achieve an initial minimum unconfined strength of 300 psi.

The exposed subgrade should be:

- Scarified to approximately 12 inches.
- Thoroughly mixed to the minimum depth of 12 to 18 inches with 3 to 7 percent cement.
- Compacted to at least 95 percent relative compaction.

Proper selection of the construction equipment is very important for the cement treating operation of the subgrade soils. Special construction equipment should be specified to scarify, add cement, mix cement, and compact the resulting mixture to the specified depths. Conventional construction equipment will likely be unable to satisfactorily complete the mixing and compacting to the required depth.

Based on our limited field exploration and laboratory testing, a summary of the subgrade soil in situ and optimum moisture contents is provided in Table 2. This provides an indication of where the soils have potentially below or above the optimum moisture contents. In the areas where soils are 3% to 4% above optimum moisture content, significant drying of the soils may be required. After drying (close to optimum or optimum plus 2% moisture), the soils can be compacted. During the fill compaction process, if any fill areas are observed to pump, the compaction process should be stopped and the pumping soils either removed and replaced as described above or disked and allowed to dry to near optimum moisture content. After drying and/or replacement, the compaction process can be restarted. The compacted fill should be firm, hard, and unyielding regardless of the relative compaction of the fill material. The finish grade should be proof rolled and soft areas should be reworked until the fill is firm, hard, and unyielding.

Fill and backfill should be compacted by:

- Placing in loose layers less than 8 inches thick.
- Moisture-conditioning to above optimum moisture content.
- Compacting to at least 90% and 95% relative compaction for fine-grained and coarse-grained soils, respectively.

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

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

Materials for structure backfill and import fill should meet the criteria in Table 7.

Table 7 - FILL AND BACKFILL CRITERIA

CRITERIA	STRUCTURE BACKFILL[1]	IMPORT FILL
Caltrans Specifications Section (Caltrans, 2018a)	19-3.02.C	19-6.02
Greenbook Specifications Section (Greenbook, 2018)	217-3	--
Maximum particle size (inches)	3	4
Maximum percentage passing the No. 200 sieve (%)	40	40
Maximum liquid limit (%)	25	30
Maximum plasticity index (%)	10	20
Minimum sand equivalent	20[2]	--
Minimum California R-value	--	50[3]

Note(s)

1. Structure backfill is material placed within the zone shown on Figure 3 and Figure 5.
2. Required behind retaining walls; within a horizontal distance of 5 feet or one-half of the wall height (whichever is greater).
3. Required only where proposed pavements are planned.

The upper granular soils encountered in the borings are expected to meet the above criteria for import fill and can be reused at the site upon proper blending and moisture conditioning.

Site grading may be accomplished with conventional heavy-duty construction equipment. However, to avoid overstressing retaining walls when placing backfill adjacent to retaining walls, backfill should be compacted using lightweight compaction equipment or the walls should be braced.

4.3.2 Excavations and Temporary and Permanent Slopes

The stability of temporary excavations is a function of several factors, including the total time the excavation is exposed, moisture condition, soil type and consistency, and contractor's operations. The contractor is responsible for excavation safety. As a guideline, temporary construction excavations greater than 3 feet but less than 10 feet deep should be planned with slopes no steeper than 1.5H:1V (horizontal to vertical). The locations for proposed steeper than 2:1 cut slopes have been evaluated from an engineering geologic perspective, provided in the Technical Memorandum in Appendix D of this Report. For the areas specified, the proposed steeper than 2:1 cut slopes should be grossly stable during excavations (Wilson Geoscience, Inc., 2019). Steeper temporary construction slopes or deeper excavations, shoring should be provided for stability and protection. The contractor should strictly adhere to grading requirements of Riverside County and applicable health and safety regulations, including those of the Occupational Safety and Health Administration (OSHA). In accordance with OSHA regulations, the near-surface, on-site soils are classified as Type C.

The shoring design should be completed by a professional Civil Engineer. Suggested minimum lateral earth pressures for use in shoring design are presented on Figure 4. Recommended maximum passive pressure resistance values are also included on Figure 4.

The type of temporary shoring selected by the contractor will depend on the contractor's proposed methods of operation, equipment, and experience. Several different types of shoring can be selected that include soldier piles and lagging, steel sheet piles, shields, and others. However, we note that braced, solid-steel sheet pile shoring is routinely used to limit the effects of excavation on adjacent structures and utilities. The sheet pile embedment depth should be determined by the contractor at the time of construction. Some difficulty in shoring installation should be anticipated, especially in dense sands.

Shoring should be checked frequently for lateral and vertical movement. If tension cracks appear in the ground surface adjacent to the shoring, the cracks should be monitored and sealed to prevent infiltration of water, and the significance of the cracks should immediately be evaluated. If large deflections (greater than 0.5% of the shoring height) are noted, the bracing systems should be checked and strengthened as needed.

Removal of the temporary shoring system should be performed using methods to prevent vibration-induced settlement of site soils.

Permanent compacted fill slopes should be planned no steeper than 2H:1V. The slopes should be paved or covered with vegetation to reduce surface erosion.

4.4 FOUNDATION DESIGN

The proposed retaining walls are planned to be supported on shallow foundations placed on a layer of compacted fill as shown on Figure 3. Because the site PGA is higher than 0.6g, in accordance with Caltrans guidelines, Caltrans standard walls cannot be used and, therefore, modified standard walls (special designed walls) should be used.

California Amendments to American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) bridge design specifications, sixth edition, updated by Caltrans to reflect the implementation of LRFD methodology for design of retaining walls (Caltrans, 2014) were used for the design of retaining walls for this Project.

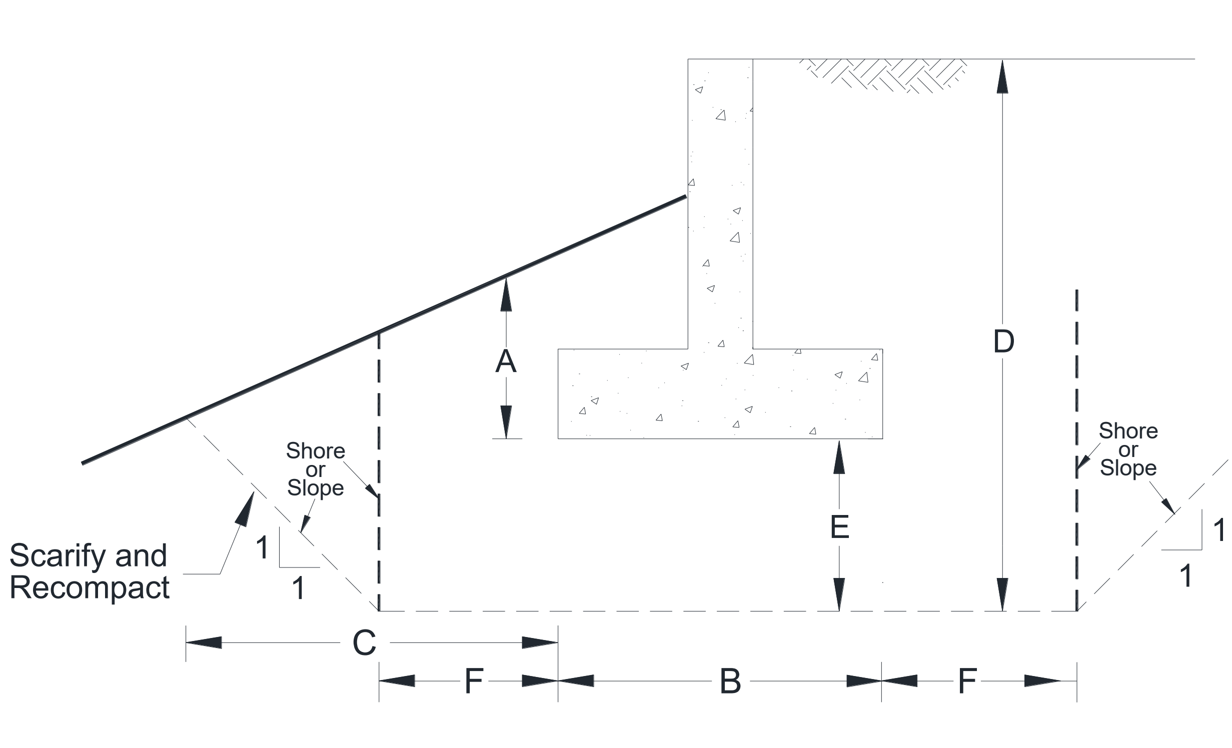
The proposed retaining walls on slopes can be supported on shallow foundations placed on a layer of compacted fill as shown on Figure 3. The factored gross nominal bearing resistances calculated based on the following are presented on Figure 3:

- Minimum embedment (toe cover)
- An effective footing width
- A 2H:1V (horizontal to vertical)
- A resistance factor of 0.45 for strength limits state demands

The permissible net contact stresses based on embedment and the effective footing widths were calculated and are presented on Figure 3.

The factored gross nominal bearing resistances should be checked by the design team against the gross uniform bearing stress. Similarly, permissible net contact stress should be checked by the design team against net bearing stress. If permissible net contact stress or factored gross nominal bearing resistances exceed the structural demand, then the geotechnical professional should be consulted.

For properly constructed foundations supported on compacted fill, total settlement due to the proposed structural loads is estimated to be less than 1 inch. Most of the settlements are expected to occur as the loads are applied or shortly thereafter. The foundation recommendations for proposed walls based on LRFD methodology are summarized on Figure 3.



LOCATION	MINIMUM VALUES		
	Wall 1	Wall 2	Wall 3
Wall height (feet)	10.5	9	15
A. Footing Embedment Below Subgrade (feet)	4	4	4
B. Footing Width (feet)	8.5	8.5	8.5
C. Minimum offset to slope[5] (feet)	4	4	4
D. Excavation Below Existing Grade (feet)	--[4]	--[4]	--[4]
E. Excavation Below Footing (feet)	2	2	2
F. Excavation Beyond Footing (feet)	1	1	1
Permissible Net Contact Stress[1,2] (ksf)	4.7	7.4	5.8
Factored Gross Nominal Bearing Resistance (Strength Limit) [3](ksf)	3.1	5.1	3.1
Factored Gross Nominal Bearing Resistance (Extreme limit event)[3](ksf)	6.8	11.3	6.8

Notes:

1. Calculated based on effective width of the footing for a total static settlement of 1 inch.
2. Permissible net contact stress and factored gross bearing nominal resistance associated with the above-noted foundation dimensions should be checked by structural engineer.
3. Factored gross nominal resistance was calculated by applying a resistance factor of 0.45 to the gross nominal bearing resistance for strength limit state and resistance factor of 1.0 for extreme event limit state.
4. Retaining wall height plus 4 feet.
5. For footings constructed on slopes, a minimum horizontal distance of 4 feet, measured at the top of the footing, shall be provided between the near face of the footing and the face of the finished slope, (Caltrans, 2003).

- ksf = kips per square foot.

Figure 3 - GRADING/FOUNDATION DETAILS

4.5 SLOPE STABILITY

Global stability analyses were conducted for static and pseudo-static conditions for the proposed retaining walls for potential deep-seated failures below the foundation. The analyses were performed using the computer program SLOPE/W (Geo-Slope International Ltd., 2008).

The soil strength parameters in Table 3 were used for the analyses. Our understanding is that the construction of the retaining walls either requires shoring or sloping back cut. As such, a slope stability analysis was performed for a back slope cut scenario (temporary condition). All existing and graded final slopes should not exceed a 2H:1V slope, unless the area was evaluated in the technical memorandum (Wilson Geoscience, Inc., 2019) provided in Appendix D. For temporary and static condition, a 2-foot soil surcharge was applied at the top of the slope to represent traffic loading. For pseudo-static condition, a horizontal seismic coefficient of 0.3g (1/3 of the site PGA) was applied.

The calculated factors of safety for deep-seated failure for different scenarios are summarized in Table 8. Analyses outputs are presented in Appendix D.

Table 8 - SUMMARY OF SLOPE STABILITY ANALYSES

		FACTOR OF SAFETY		
			Permanent	
STRUCTURE	STATION	Temporary Static	Static	Pseudo-Static
RW1	344+00	1.4	1.7	1.1
	344+50	1.6	1.6	1.0
RW2	347+00	1.6	1.8	1.1
RW3	349+50	1.3	2.0	1.3
Note: See Appendix D for slope stability outputs.				

For all graded slopes, proper maintenance with erosion protection and drainage control in accordance with Section 21 of Caltrans Standard Specifications (Caltrans, 2018a) are recommended.

4.6 RESISTANCE TO LATERAL LOADS AND LATERAL EARTH PRESSURES

4.6.1 Temporary Structures

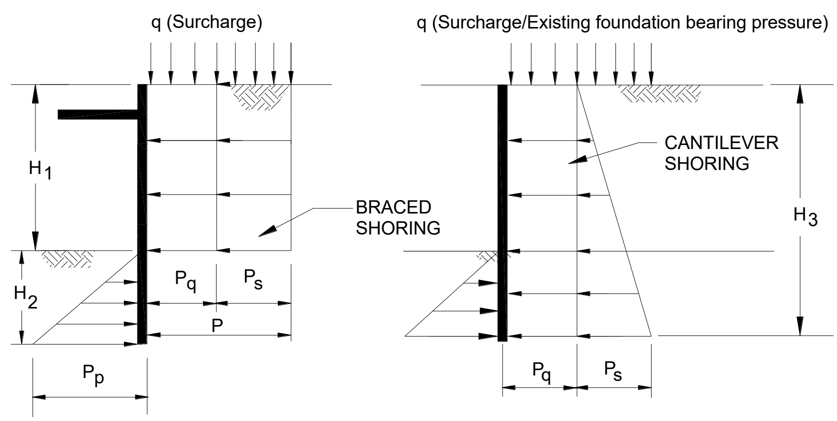
To avoid potential conflicts, shoring should be carefully considered based on shoring depth and proximity to the existing active travel lanes. Suggested lateral earth pressures for use in shoring design are presented on Figure 4, which also includes the effect of general surcharge and traffic.

We recommend that the design of temporary shoring be performed using shoring pressures equal to or greater than those shown on Figure 4 and that the passive resistance be equal to or less than that shown on Figure 4.

An allowable passive soil pressure as outlined on Figure 4 can resist lateral loads. The passive pressures on Figure 4 assume either undisturbed natural soils or compacted fill. The upper 1 foot of passive resistance should be neglected unless confined by a pavement or slab-on-grade.

Shoring should be checked frequently for lateral and vertical movement. If tension cracks appear in the ground surface adjacent to the shoring, the cracks should be monitored and sealed to prevent infiltration of water, and the significance of the cracks should be evaluated immediately. If large deflections (greater than 0.5% of the shoring height) are noted, the bracing systems should be checked and strengthened as needed.

Removal of the temporary shoring system (if necessary) should be performed very carefully to prevent vibration-induced settlement of site soils.



P_P = 340 H_2 psf	Braced Shoring P = P_s + P_q = 25 H_3 + 0.5q = (300 psf minimum)	Cantilever Shoring P = P_s + P_q = 38 H_3 + 0.33q = (300 psf minimum)

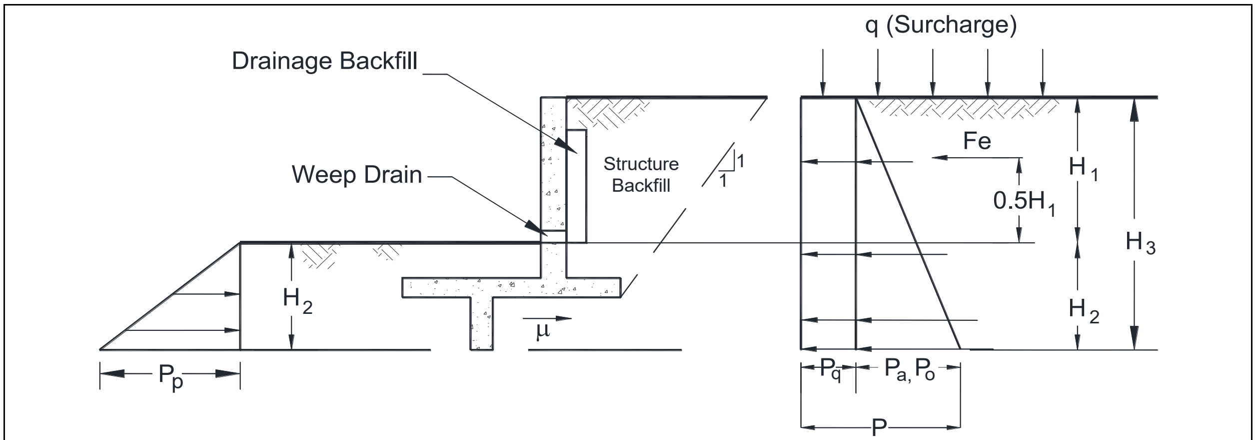
Note(s):

- All values of height (H) in feet; pressure (P) and surcharge (q) in psf.
- Value for temporary excavations.
- For traffic loads, assume a uniform 100 psf horizontal pressure over the top 10 feet of wall.

Figure 4 - LATERAL EARTH PRESSURE - TEMPORARY SHORING

4.6.2 Permanent Structures

The proposed retaining walls should be designed to resist lateral earth pressures with the equivalent fluid pressures as illustrated on Figure 5 . Lateral earth pressures are presented for walls free to rotate, which is applicable to the Project. The lateral earth pressures on Figure 5 are based on structure backfill per Caltrans Standard Specifications (Caltrans, 2018a).



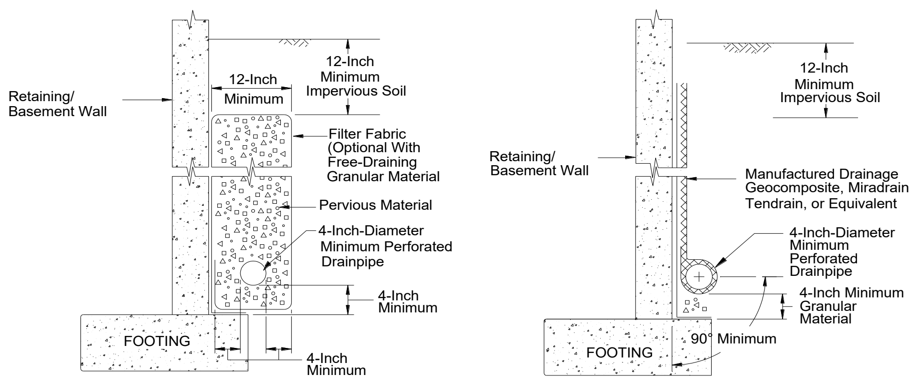
$P_p = 360\ H_2$ $\mu = 0.6$	Cantilever Walls $P = P_a + P_q = 36\ H_3 + 0.3q$ $Fe = 27\ H_1^2$

Note(s):

- All values of height (H) in feet, pressure (P) and surcharge (q) in pounds per square foot (psf), and force (F) in pounds.
- P_p, P_a, and P_o are the passive, active, and at-rest earth pressures, respectively; Fe is the incremental seismic force.
- Horizontal and vertical inertia forces should be used per AASHTO LRFD Bridge Design Specification.
- The resistance to sliding (R_R) can be estimated using $R_R = \Psi_{ep}R_{ep} + \Psi_t R_t$; where Ψ_{ep} = resistance factor for passive resistance, Ψ_t = resistance factor for shear (sliding), R_{ep} = nominal passive resistance, and R_t = nominal sliding resistance. Use P_p and μ provided above for estimation of R_{ep} and R_t.
- Ψ_{ep} = 0, 0.5, 1 for service, strength, and extreme events, respectively.
- Ψ_t = 1, 0.8, 1 for service, strength, and extreme events, respectively.
- P_q is the incremental surcharge pressure, and μ is the friction coefficient applied to dead normal (buoyant) loads. Fe is in addition to the active and at-rest pressures. Below groundwater, active and at-rest pressure should be reduced by 50% and hydrostatic pressure should be added to active and at-rest pressures. P_p should be reduced by 50% below the groundwater.
- For 2H:1V slopes above the wall, increase the active and at-rest pressures by 50%; for 1.5H:1V slope, increase the active and at-rest pressures by 100%.
- Where the foundations are located on a slope, if the set back to the edge of the foundation from the slope surface is at least 5 feet or if the slope is no steeper than 4H:1V, no reduction in passive pressures are needed. Otherwise, reduce the passive pressures by 60% for a 2H:1V slope and by 40% for a 3H:1V slope.
- Neglect the upper 1 foot for passive pressure unless the surface is contained by a pavement or slab.
- Fifty percent (50%) of PGA was used to calculate Fe.

Figure 5 - LATERAL EARTH PRESSURES – PERMANENT STRUCTURES

Retaining walls should be designed to resist lateral earth pressures with the equivalent fluid pressures as illustrated on Figure 5. Lateral earth pressures are presented for walls free to rotate. See Figure 6 for typical sections of wall drains.



MATERIAL	CALTRANS SPECIFICATIONS	GREENBOOK SPECIFICATIONS
Free-Draining Granular Material	68-2.02 (Class 2)	300-3.5.2
Geotextile Filter Fabric	96-1.02B	300-8
Perforated Pipe	68-2.02	207-13.4

Note(s):
- Drainpipe should drain to an outlet.
- Filter fabric wraps completely around perforated drainpipe and pervious materials.

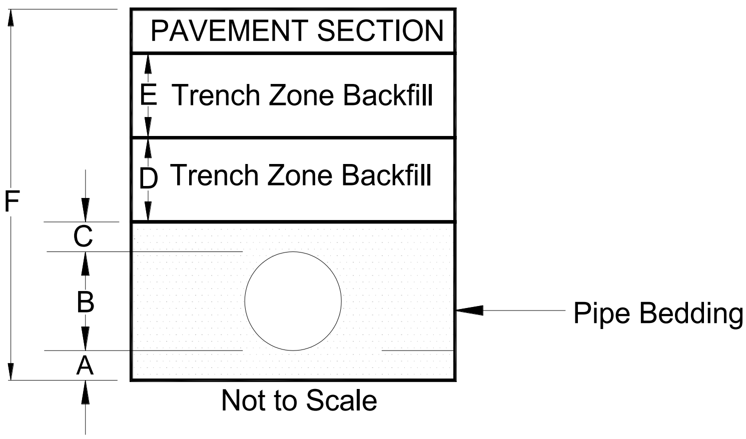
Figure 6 - RETAINING OR BASEMENT WALL DRAINAGE

4.7 UTILITY TRENCHES

Utility trenches (either open or backfilled) that parallel structures, pavement, or flatwork should be planned so that they do not extend below a plane with a downward slope of 1.5H:1V from the bottom edge of footings, pavement, or flatwork. Temporary shoring to provide footing, pavement, flatwork, or utility support is recommended unless localized settlements on the order of 1% of the trench depth can be tolerated.

All excavations should comply with appropriate safety standards outlined in Section 4.3.2.

Utility pipes should be placed on the bottom of a neatly cut trench on a layer of bedding as outlined on Figure 7 or according to the manufacturer's recommendations, whichever is greater. Jetting should not be allowed for compaction purposes. We anticipate that the near-surface sandy soils will be suitable for use as backfill. The near-surface sandy soils should be tested to check whether they meet the criteria for bedding soils.



MATERIAL	MINIMUM THICKNESS (feet)	MINIMUM RELATIVE COMPACTION[1] (%)	BACKFILL SPECIFICATIONS	
			Caltrans Specifications	Greenbook Specifications
Pipe Bedding	A = 0.33 or B/4	--	19-3.02E	306-6
Pipe Zone	C = 1	--	19-3.02E	306-6
Trench Zone	D varies	90[2]	--	--
Trench Zone[3]	E = 2	95	--	--

Note(s):

1. Based on ASTM D1557.
2. To reduce settlement, use 95% relative compaction.
3. E = 0 if no pavement or settlement-sensitive structures at surface.

- Minimum values; use manufacturer's recommendations if greater.
- For slurry backfill, Caltrans Specification 19-3.02D can be used.
- Bottom of the trench should be checked by a geotechnical professional prior to placing pipe bedding. The bottom should be firm, hard and unyielding.

Figure 7 - PIPELINE BACKFILL SCHEMATIC

4.8 PAVEMENT SECTIONS

The recommend minimum pavement thicknesses (flexible and rigid) presented on Figure 8 is based on the following:

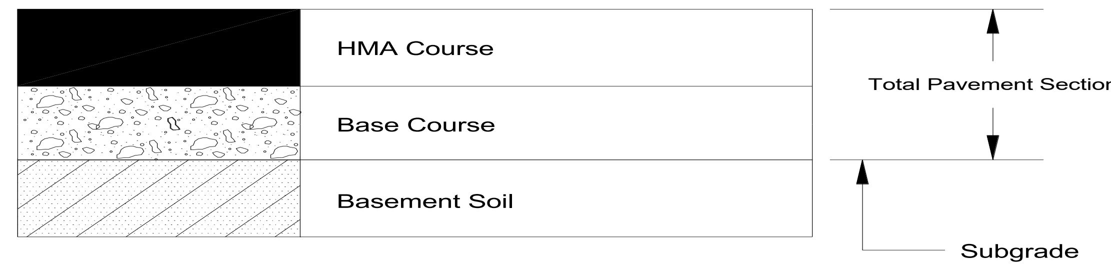
- Caltrans design method (2017a, b).
- California R-values of 20, 30, and 50 of on-site subgrade soils. California R-value of 30 for the cement treated subgrade soils.
- Subgrade Types I & II and Inland Valley Pavement Climate Region.
- Traffic index (TI) of 9.5 provided by County of Riverside.
- Table 9 summarizes stations and corresponding soil Types and R values.

The minimum thickness of compacted basement soil and aggregate base (AB) are outlined on Figure 8. The basement soils should be firm, hard, and unyielding, and not "pumping" prior to placing the AB. The AB requirements and specifications are outlined on Figure 8. If the basement soil cannot be compacted, the soil should be overexcavated as noted in Section 4.3.

Table 9 – SUMMARY OF STATIONS, R-VALUE AND SUBGRADE TYPE

STATION	R-VALUE	SUBGRADE TYPE[1]
179+50 – 195+00	50	I
195+00 – 212+00	20 (30)[2]	II
212+00 – 250+00	50	I
250+00 – 300+00	20 (30)[2]	II
300+00 – 375+00	50	I
375+00 – 405+40	20 (30)[2]	II

Notes:
1. Type I – Silty Sand (SM); Type II – Lean Clay (CL)
2. We judge that the subgrade soils can be treated with 3 to 7% of cement to achieve R-value of 30.
- For stations, see Project Plans in Appendix A.



COURSE	MINIMUM THICKNESS[1] (feet)										
	RHMA/HMA/AB			HMA/AB			RHMA/HMA			JPCP	
	RV = 20	RV = 30[5]	RV = 50	RV = 20	RV = 30[5]	RV = 50	RV = 20	RV = 30[5]	RV = 50	Subgrade Type I	Subgrade Type II
RHMA[2]	0.2	0.2	0.2	--	--	--	0.2	0.2	0.2	--	--
HMA/JPCP[3]	0.5	0.3	0.3	0.7	0.5	0.5	0.95	0.8	0.7	0.95	0.95
Base[4]	1.1		0.6	1.1		0.6	--			0.60	1.0
Basement Soil[6]	1.0		1.0	1.0		1.0	1.0			1.0	1.0

Note(s):

1. Pavement sections based on TI = 9.5
2. Rubberized hot mix asphalt (RHMA) should satisfy the requirements of the Standard Specifications for Public Works Construction (Greenbook) Sections 203 and 302 (Building News, 2018).
3. Hot mix asphalt (HMA) and jointed plain concrete pavement (JPCP) should satisfy the requirements of Caltrans Standard Specifications Sections 39 and 40 (Caltrans, 2018a), respectively, or Greenbook Sections 203 and 302, and 201 and 302 (Building News, 2018), respectively.
4. Base course = Type II AB or crushed miscellaneous base (CMB), in accordance with Caltrans Standard Specifications Section 26 (Caltrans, 2018a) or Greenbook Sections 200-2.4 (Building News, 2018), respectively. The minimum relative compaction is 95% in accordance with ASTM D1557.
5. Subgrade soil should be treated with cement (approximately 3 – 7% by dry weight) to achieve the minimum R-Value of 30.
6. Compacted-in-place natural basement soil or fill; at least 95% relative compaction.

- RV = California R-Value

Figure 8 - PAVEMENT STRUCTURAL SECTION

4.9 CORROSION POTENTIAL

Soil samples were collected at 6 borings, and 8 bulk sample locations and tested for pH, soluble chloride and soluble sulfate, and soil electrical resistivity for corrosion potential. The range of test values is summarized in Table 10.

Also presented in Table 10 are Caltrans (2018b) corrosion criteria. The corrosion potential test results are presented in Appendix B. Based on Caltrans standards and the chemical test results, the on-site soils are classified as non-corrosive to buried metal pipes. Borrow soils imported to the Project site should be tested for corrosion potential.

Table 10 - CORROSION POTENTIAL TEST SUMMARY

BORING	DEPTH (bgs)	pH	SOLUBLE SULFATE CONTENT (ppm)	SOLUBLE CHLORIDE CONTENT (ppm)	ELECTRICAL RESISTIVITY (ohm-cm)
DYB19-03	0 - 5	8.1	86	57	1,608
DYB19-06	1 - 5	8.5	187	42	1,340
DYB19-09	2.5 - 15	8.6	48	12	4,221
DYB19-10	2.5 – 13	9.1	9	1	9,380
DYB19-11	2.5 - 17	9.1	16	4	7,370
DYB19-13[1]	0 - 5	9.2	270	472	**670**
C-01	0 - 2	8.2	16	1	5,896
C-02	0 - 2	8.4	3	1	4,891
C-03	0 - 2	7.8	33	3	1,675
C-04	0 - 2	8.6	28	1	2,345
C-05	0 - 2	8.1	7	3	1,943
C-06	0 - 2	7.8	6	2	1,876
C-07	0 - 2	7.3	7	3	2,211
C-08	0 - 2	8.0	12	2	2,881

Note:

1. Agencies such as Los Angeles County would consider the soil from 0 to 5 feet at DYB19-13 to be corrosive because the electrical resistivity was not > 1,100 ohm-cm.

- For structural elements, Caltrans (2018b) considers a site to be corrosive if one or more of the following conditions exist for the representative soil sample taken from the site:
  - Chloride concentration is 500 ppm or greater
  - Sulfate concentration is 1,500 ppm or greater
  - pH is 5.5 or less
- Resistivity is not included as a parameter to define a corrosive area for structures; however, in general, the higher the resistivity, the lower the rate for corrosion. A minimum resistivity value for soil less than 1,100 ohm-cm indicates the presence of high quantities of soluble salts and a higher propensity for corrosion.

5 PLAN REVIEW, CONSTRUCTION OBSERVATION, AND TESTING

DYA should review the final grading plans, foundation plans, and geotechnical related specifications for conformance with the intent of our recommendations. The review will enable DYA to modify the recommendations if final design conditions are different than presently understood.

During construction, DYA should provide field observation and testing to check that the site preparation, excavation, foundation installation, and finished grading conform to the intent of these recommendations, project plans, and specifications. This would allow DYA to develop supplemental recommendations as appropriate for the actual soil conditions encountered and the specific construction techniques used by the contractor.

As needed during construction, DYA should be retained to consult on geotechnical questions, construction problems, and unanticipated site conditions.

6 RESPONSE TO REVIEW COMMENTS

We have reviewed the comments made by the Riverside County Project Team (Alfredo Martinez and Elmer Datuin) in August and September of 2019. Our responses to the review comments are provided in Appendix F.

7 LIMITATIONS

This report has been prepared for this Project in accordance with generally accepted geotechnical engineering practices common to the local area. No other warranty, expressed or implied, is made.

The analyses and recommendations contained in this report are based on the literature review, limited field exploration, and limited laboratory testing conducted in the area. The results of the field exploration indicate subsurface conditions only at the specific locations and times, and only to the depths penetrated. They do not necessarily reflect strata variations that may exist between such locations. Although subsurface conditions have been explored as part of the exploration, we have not conducted chemical laboratory testing on samples obtained or evaluated the site with respect to the presence or potential presence of contaminated soil or groundwater conditions, for mold, or methane gas.

The validity of our recommendations is based in part on assumptions about the stratigraphy. Observations during construction can help confirm such assumptions. If subsurface conditions different from those described are noted during construction, recommendations in this report must be re-evaluated. DYA should be retained to observe earthwork construction in order to help confirm that our assumptions and recommendations are valid or to modify them accordingly. DYA cannot assume responsibility or liability for the adequacy of recommendations if we do not observe construction.

This report is intended for use only for the project described. In the event that any changes in the nature, design, or location of the facilities are planned, the conclusions and recommendations contained in this report should not be considered valid unless the changes are reviewed and conclusions of this report modified or verified in writing by DYA. We are not responsible for any claims, damages, or liability associated with the interpretation of subsurface data or reuse of the subsurface data or engineering analyses without our express written authorization.

8 REPORT LOG

Version	DATE	DESCRIPTION
V1	07/18/2019	Draft Report to NCM for review and comment
V2	09/01/2020	Response to comments from County
V3	10/06/2020	Final Version

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APPENDIX A -
PROJECT PLANS AND SITE PLANS


BEGIN
PHASE 4A
179+50
GILMAN SPRINGS RD
(SLOPE)
DYB19-01
C-1
SEE MATCH LINE BELOW
195+50 "G" LINE
SEE MATCH LINE ABOVE
195+50 "G" LINE
CHANDLER AGGREGATE
DYB19-02
SEE MATCH LINE SHEET 2
212+50 "G" LINE
Geotechnical Investigation Legend
DYB19-01 through DYB19-14 are Geotechnical Soil Borings
C-1 through C-9 are bulk samples for corrosion testing
LEGEND
EXISTING R/W
EXISTING EASMENT (TYPE)
PACEL LINE
SLOPE EASEMENT
DRAINAGE/UTILITY EASEMENT
TEMP CONST EASEMENT
CL
EP
0
40
80
120
SCALE: 1" = 40'
ENGINEERING COMPANY
NCM ENGINEERING CORP
4740 GREEN RIVER ROAD
SUITE 218
CORONA, CA 92880
PREPARED BY:
R.C.E. NO.
DATE
GILMAN SPRINGS RD
WIDENING
SHEET 1 OF 7

SLEGERS ST
(SLOPE)
SEE MATCH LINE SHEET 1 212+50 "G" LINE
213
214
215
216
217
218
219
GILMAN SPRINGS RD
220
221
222
223
224
225
226
227
228
229
DYB19-03
SEE MATCH LINE BELOW 229+50 "G" LINE
(DRAINAGE)
MINOR GRADING
BRIDGE ST
BRIDGE ST
SEE MATCH LINE ABOVE 229+50 "G" LINE
230
231
232
233
234
235
236
237
238
239
240
241
242
243
244
245
246
(oh)
SEE MATCH LINE SHEET 3 246+50 "G" LINE
LEGEND
EXISTING R/W
EXISTING EASMENT (TYPE)
PACEL LINE
SLOPE EASEMENT
DRAINAGE/UTILITY EASEMENT
TEMP CONST EASEMENT
CL
EP
Geotechnical Investigation Legend
DYB19-01 through DYB19-14 are Geotechnical Soil Borings
C-1 through C-9 are bulk samples for corrosion testing
0 40 80 120
SCALE: 1" = 40'
ENGINEERING COMPANY
NCM ENGINEERING CORP
4740 GREEN RIVER ROAD
SUITE 218
CORONA, CA 92880
PREPARED BY:
R.C.E. NO.
DATE
GILMAN SPRINGS RD WIDENING
SHEET 2 OF 7

SEE MATCH LINE SHEET 2 246+50 "G" LINE
DYB19-04
C-2
MIDDLETON ST
(SLOPE)
GILMAN SPRINGS RD
SEE MATCH LINE BELOW 263+50 "G" LINE
SEE MATCH LINE ABOVE 263+50 "G" LINE
DYB19-05
EAST DR
(SLOPE)
GILMAN SPRINGS RD
(DRAINAGE)
C-3
PEPPER TREE LN
SEE MATCH LINE SHEET 4 280+50 "G" LINE
SLOPE AND UTILITY EASEMENT
Geotechnical Investigation Legend
DYB19-01 through DYB19-14 are Geotechnical Soil Borings
C-1 through C-9 are bulk samples for corrosion testing
LEGEND
EXISTING R/W
EXISTING EASMENT (TYPE)
PACEL LINE
SLOPE EASEMENT
DRAINAGE/UTILITY EASEMENT
TEMP CONST EASEMENT
CL
EP
SCALE: 1" = 40'
ENGINEERING COMPANY
NCM ENGINEERING CORP
4740 GREEN RIVER ROAD
SUITE 218
CORONA, CA 92880
PREPARED BY:
R.C.E. NO.
DATE
GILMAN SPRINGS RD WIDENING
SHEET 3 OF 7

Geotechnical Investigation Legend

DYB19-01 through DYB19-14 are Geotechnical Soil Borings

C-1 through C-9 are bulk samples for corrosion testing

LEGEND

- EXISTING R/W
- EXISTING EASMENT (TYPE)
- PACEL LINE
- SLOPE EASEMENT
- DRAINAGE/UTILITY EASEMENT
- TEMP CONST EASEMENT
- CL
- EP

0 40 80 120

SCALE: 1" = 40'

ENGINEERING COMPANY
NCM ENGINEERING CORP
4740 GREEN RIVER ROAD
SUITE 218
CORONA, CA 92880
PREPARED BY: R.C.E. NO. DATE

GILMAN SPRINGS RD WIDENING

SHEET 4 OF 7

SEE MATCH LINE SHEET 4 314+50 "G" LINE
CENTRAL AVE
GILMAN SPRINGS RD
EXIST R = 1312.34'
C-6
(SLOPE)
SEE MATCH LINE BELOW 331+50 "G" LINE
SEE MATCH LINE ABOVE 331+50 "G" LINE
KNOCH RD
CURTIS ST
(SLOPE)
JACK RABBIT TRAIL
DYBI9-08 (R/W)
DYBI9-09
DYBI9-10
343+56.26 BEG RET WALL
345+00.00 END RET WALL
346+50.00 BEG RET WALL
347+50.00 END RET WALL
SEE MATCH LINE SHEET 6 348+50 "G" LINE
AERIAL OH POWER LINE EASEMENT
UTILITY EASEMENT
LEGEND
EXISTING R/W
EXISTING EASMENT (TYPE)
PACEL LINE
SLOPE EASEMENT
DRAINAGE/UTILITY EASEMENT
TEMP CONST EASEMENT
CL
EP
Geotechnical Investigation Legend
DYB19-01 through DYB19-14 are Geotechnical Soil Borings
C-1 through C-9 are bulk samples for corrosion testing
0 40 80 120
SCALE: 1" = 40'
ENGINEERING COMPANY
NCM ENGINEERING CORP
4740 GREEN RIVER ROAD
SUITE 218
CORONA, CA 92880
PREPARED BY:
R.C.E. NO.
DATE
GILMAN SPRINGS RD WIDENING
SHEET 5 OF 7


GILMAN SPRINGS RD
SEE MATCH LINE SHEET 5 348+50 "G" LINE
348+90.00 BEG RET WAL
351+60.00 END RET WALL
DYB19-11
DYB19-12
C-7
(SLOPE)
(oh)
SEE MATCH LINE BELOW 366+50 "G" LINE
SEE MATCH LINE ABOVE 366+50 "G" LINE
SEE MATCH LINE SHEET 7 383+50 "G" LINE
QUIAL RANCH GOLF COURSE
LEGEND
EXISTING R/W
EXISTING EASMENT (TYPE)
PACEL LINE
SLOPE EASEMENT
DRAINAGE/UTILITY EASEMENT
TEMP CONST EASEMENT
CL
EP
Geotechnical Investigation Legend
DYB19-01 through DYB19-14 are Geotechnical Soil Borings
C-1 through C-9 are bulk samples for corrosion testing
SCALE: 1" = 40'
0 40 80 120
NCM ENGINEERING CORP
4740 GREEN RIVER ROAD
SUITE 218
CORONA, CA 92880
GILMAN SPRINGS RD WIDENING
SHEET 6 OF 7

LEGEND

- EXISTING R/W
- EXISTING EASMENT (TYPE)
- PACEL LINE
- SLOPE EASEMENT
- DRAINAGE/UTILITY EASEMENT
- TEMP CONST EASEMENT
- CL
- EP

Geotechnical Investigation Legend

DYB19-01 through DYB19-14 are Geotechnical Soil Borings

C-1 through C-9 are bulk samples for corrosion testing

0 40 80 120

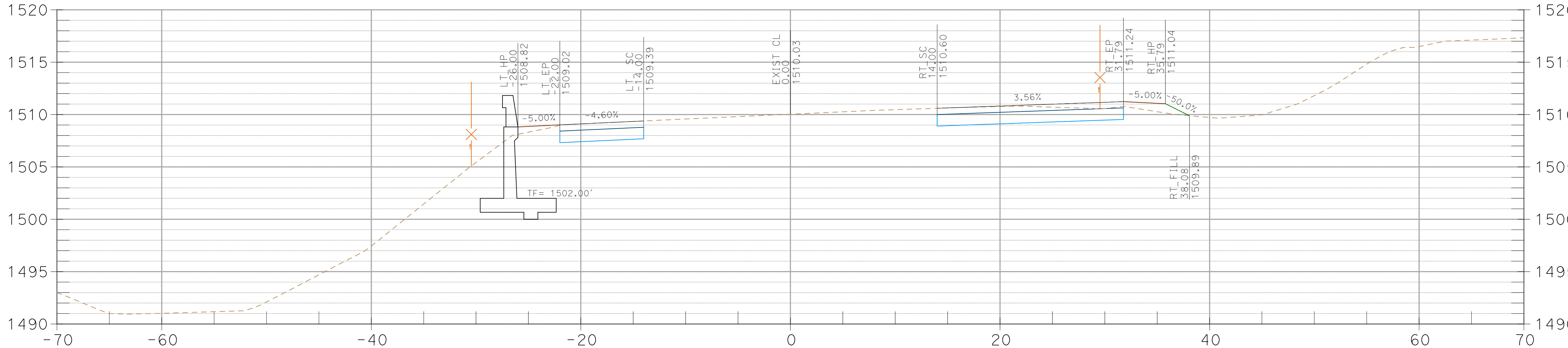
SCALE: 1" = 40'

ENGINEERING COMPANY
NCM ENGINEERING CORP
4740 GREEN RIVER ROAD
SUITE 218
CORONA, CA 92880

PREPARED BY: R.C.E. NO. DATE

GILMAN SPRINGS RD WIDENING

SHEET 7 OF 7



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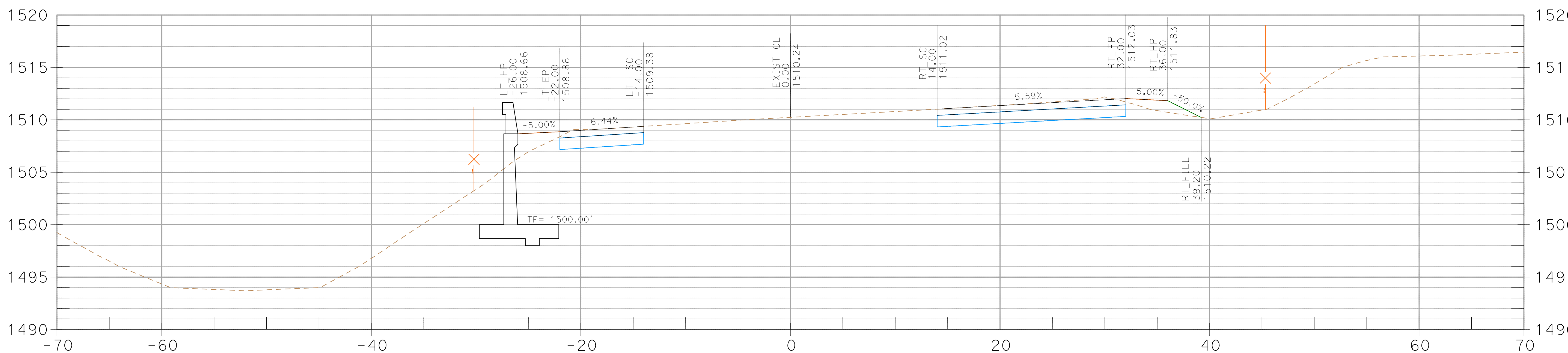

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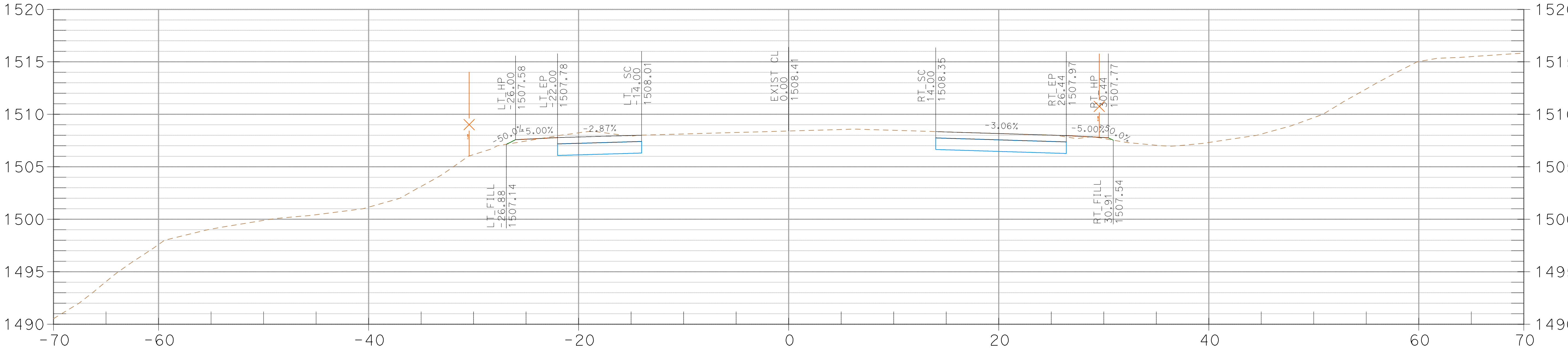

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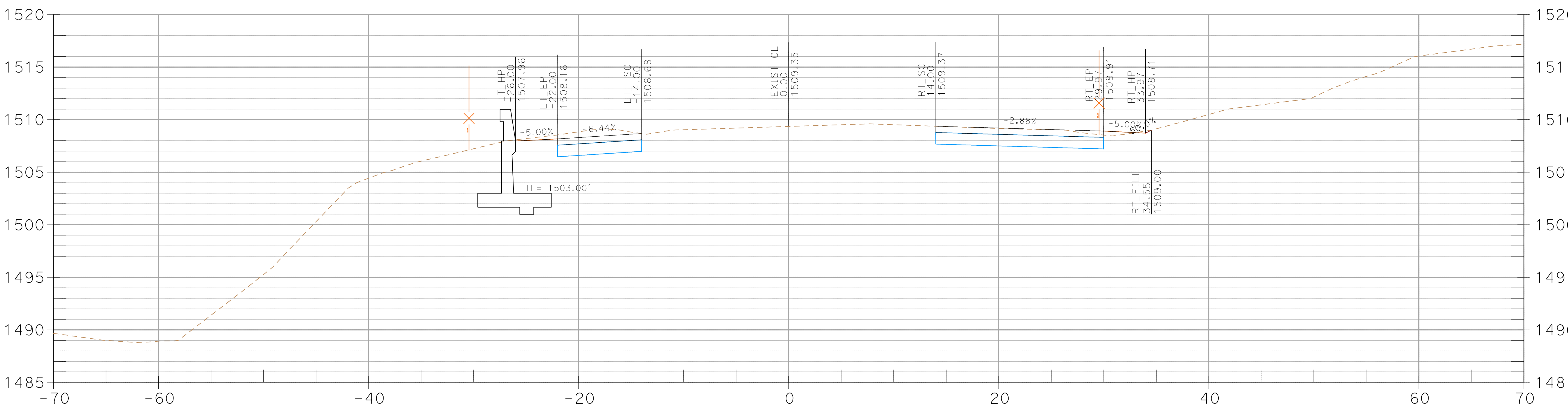

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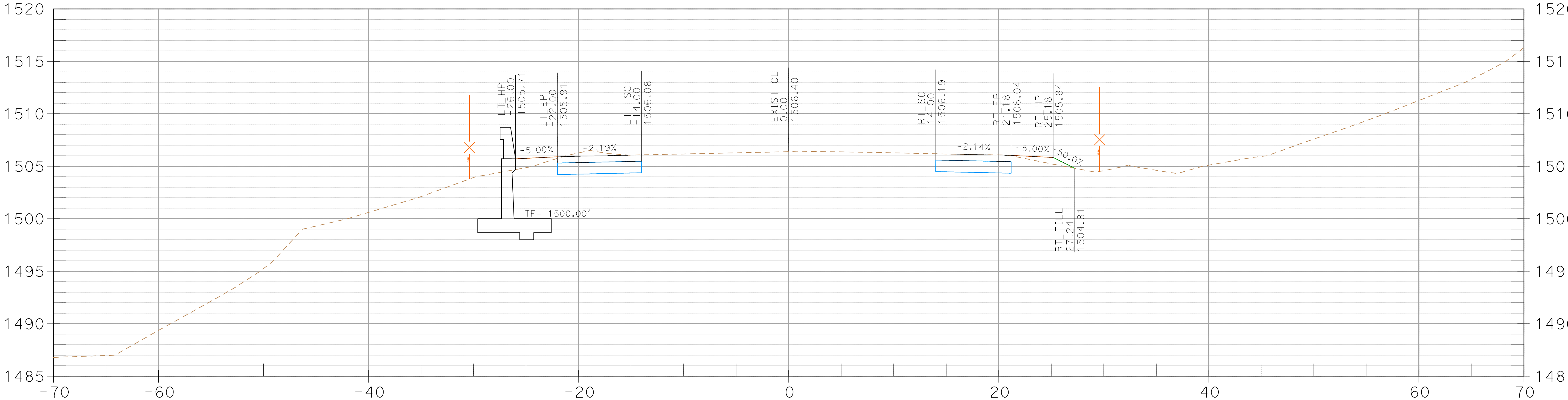

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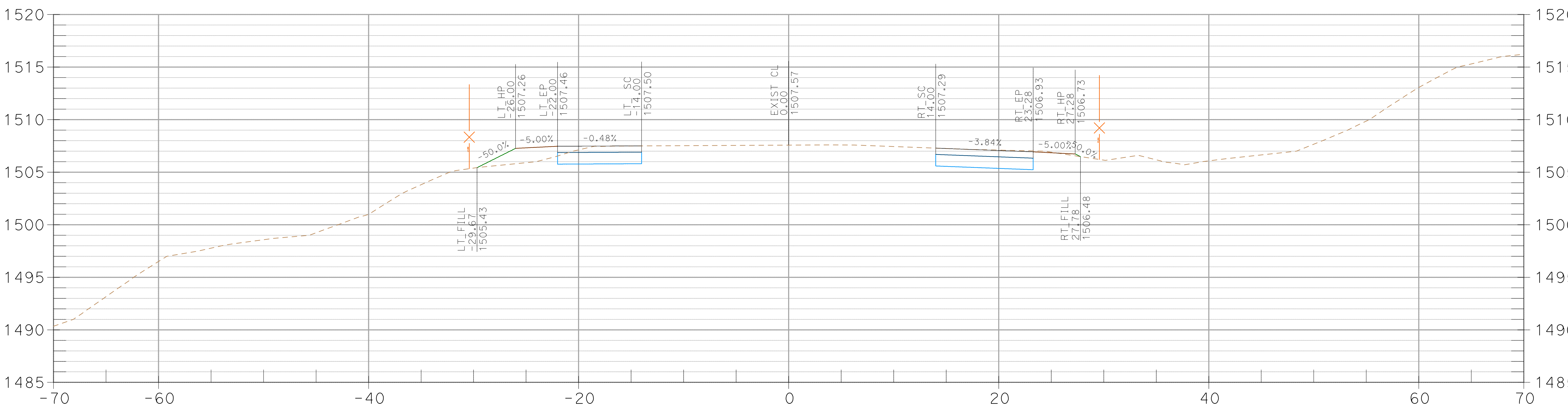

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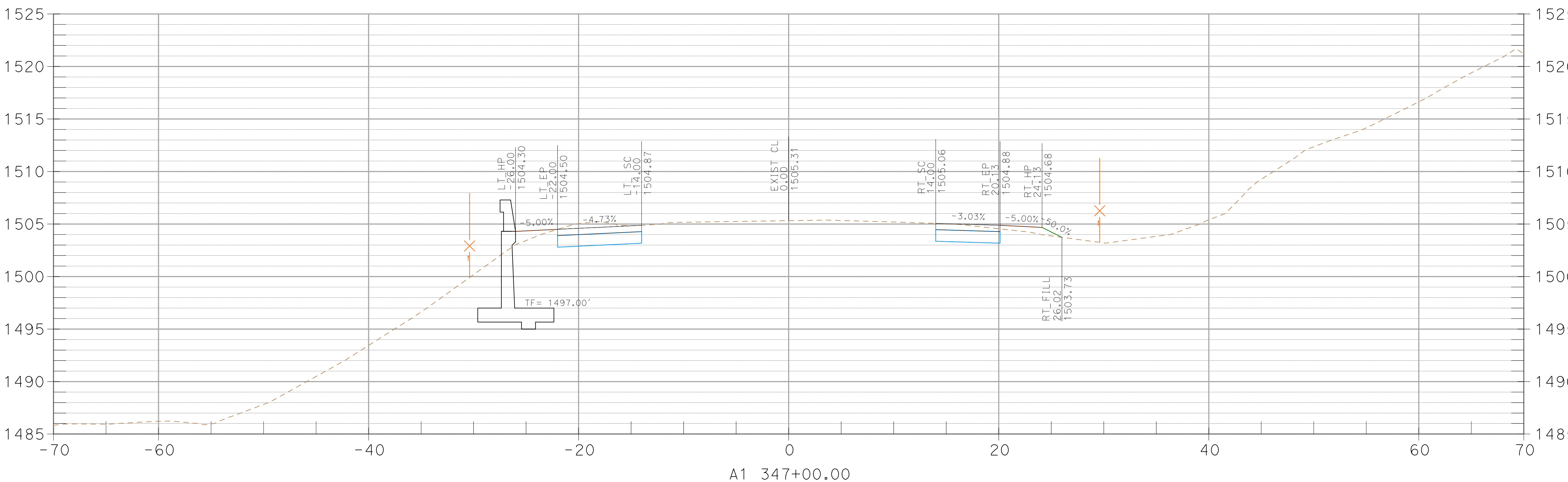

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RT_SC
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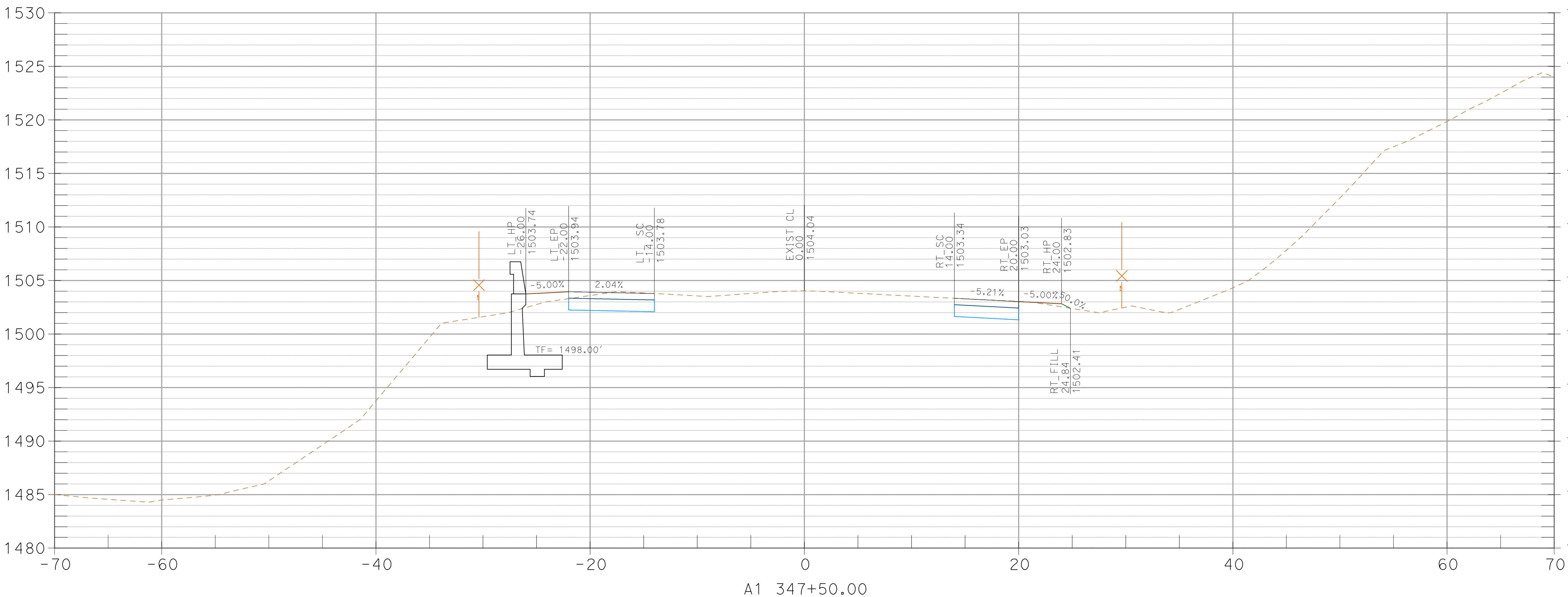
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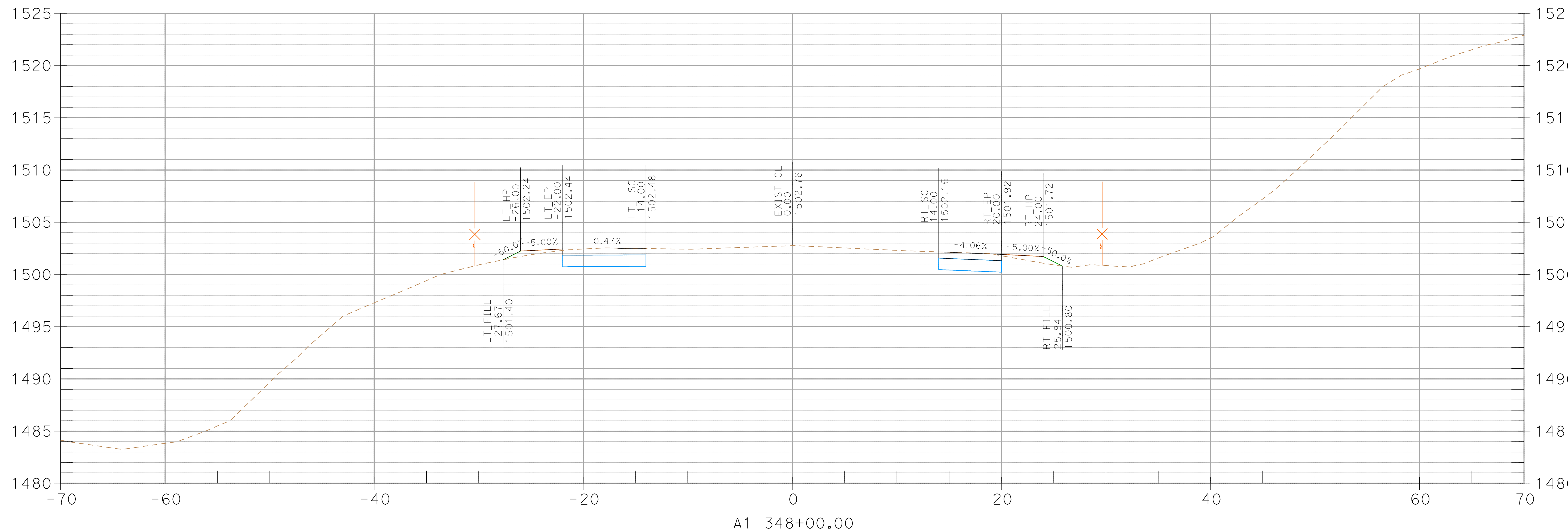
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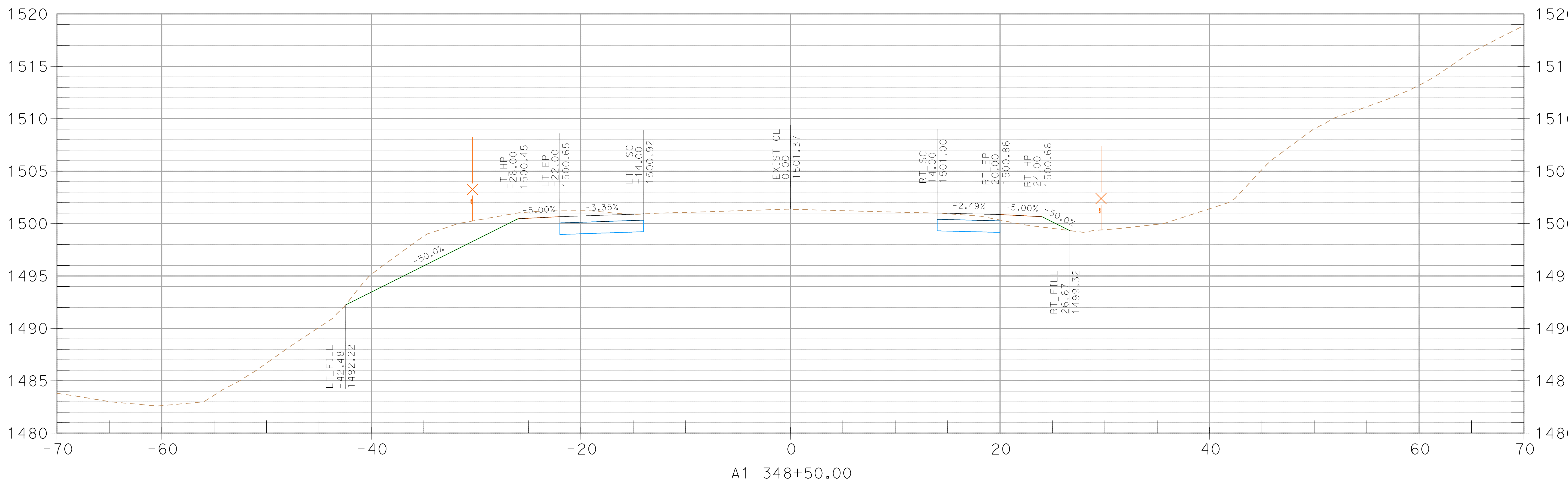
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A1 348+50.00
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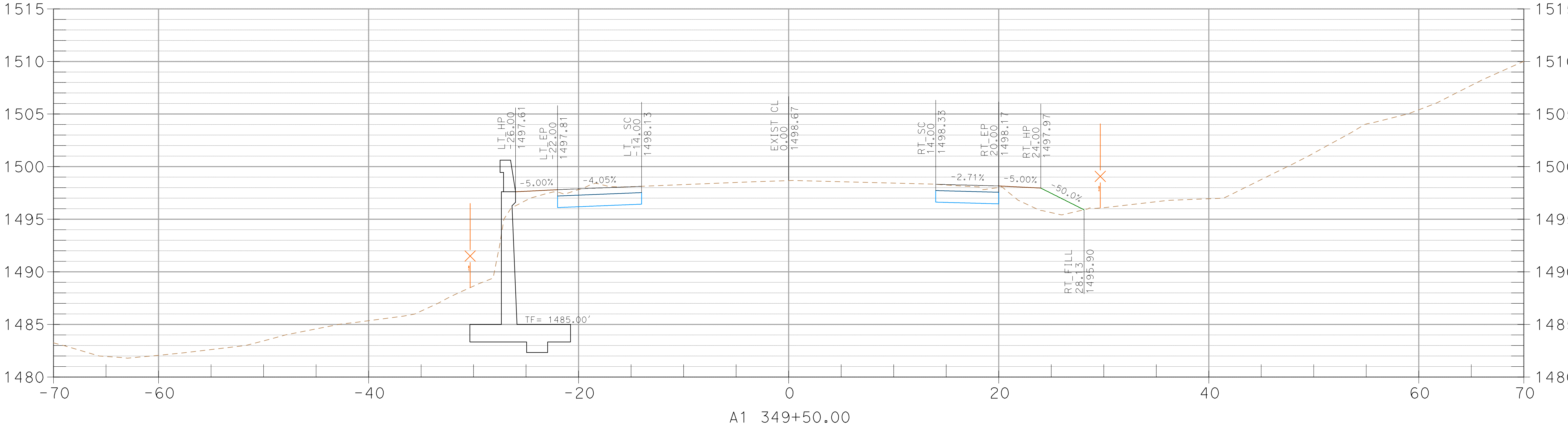
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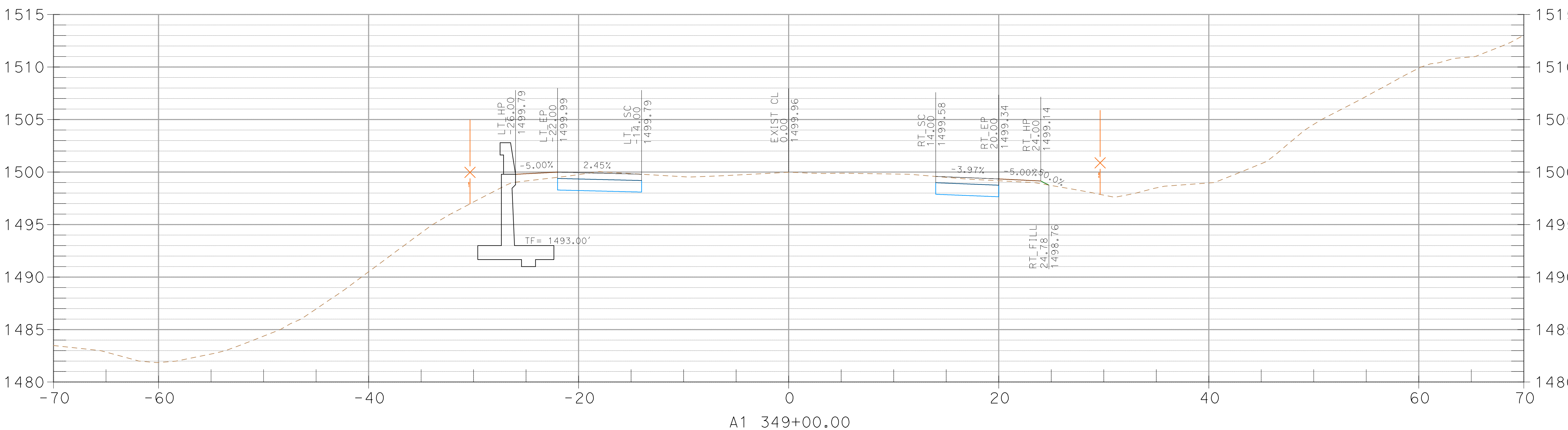
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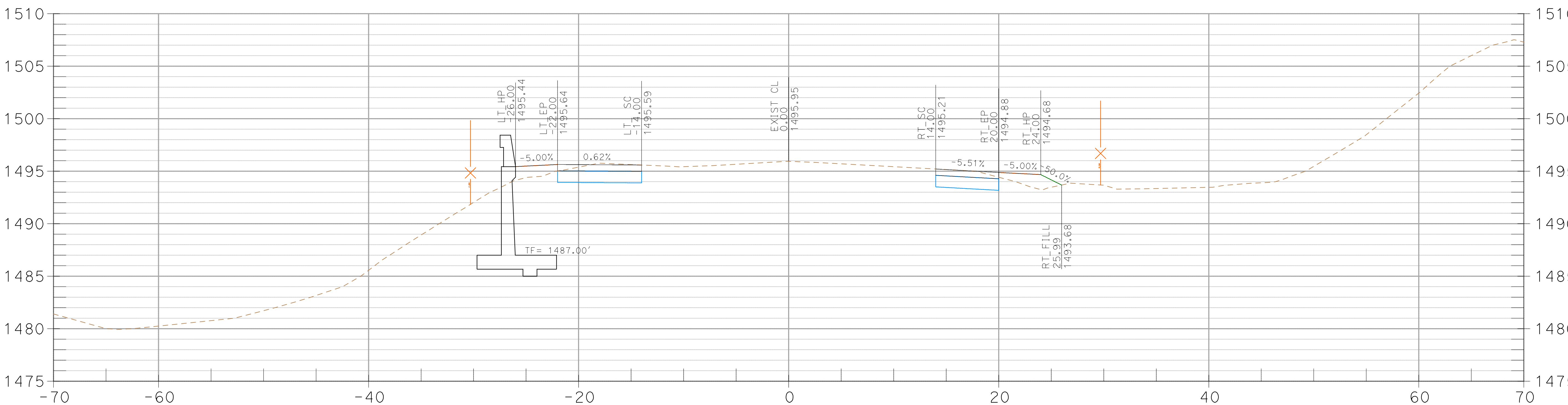

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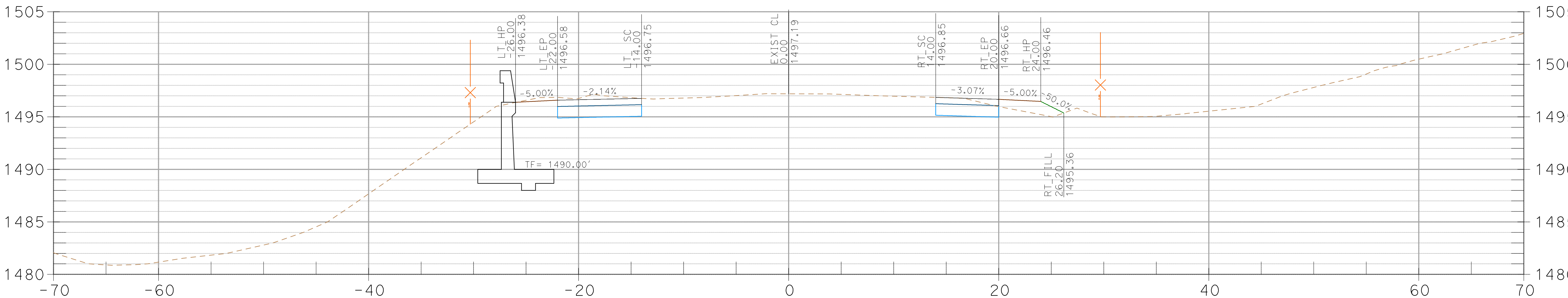

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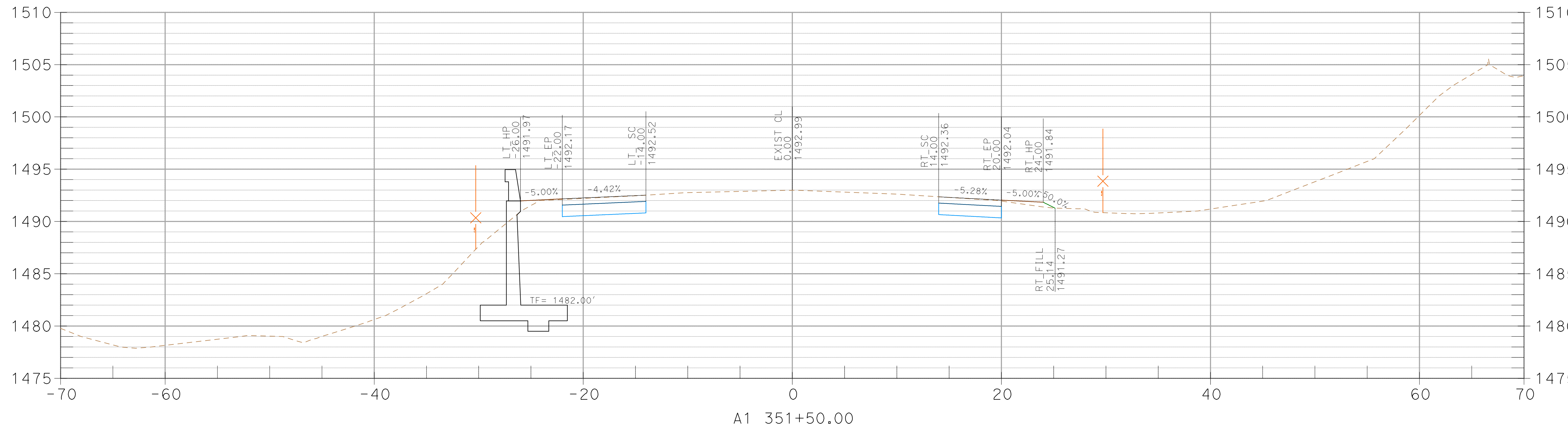
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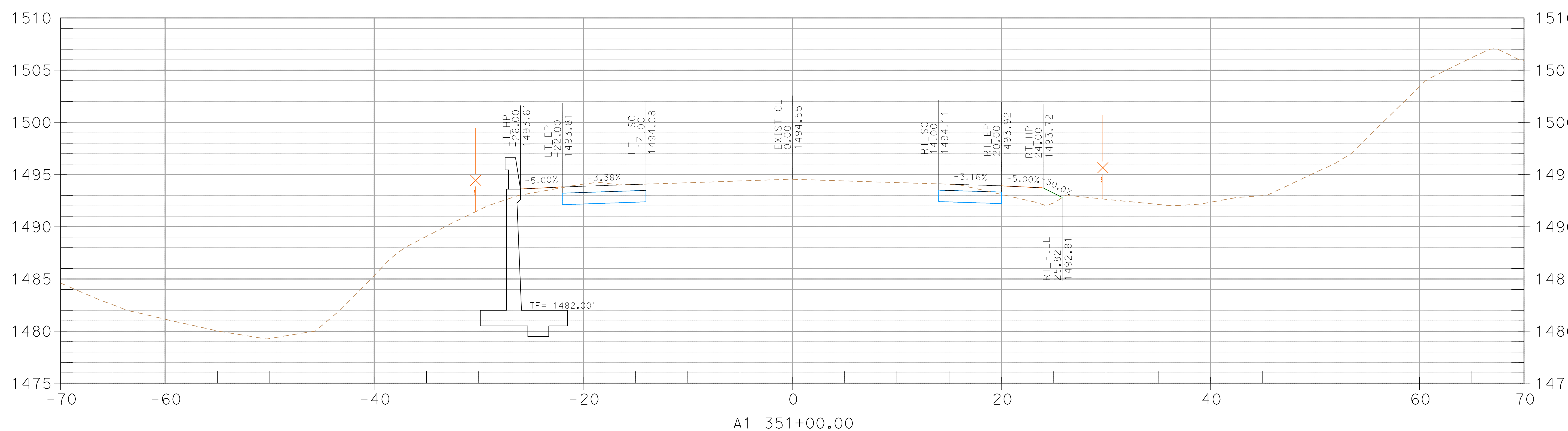
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APPENDIX B -
FIELD EXPLORATION

APPENDIX B - FIELD EXPLORATION

The field exploration for the proposed Project consisted of drilling fourteen borings (DYB19-01 through DYB19-14) and collecting eight near surface samples for corrosion tests. The approximate borings and corrosion samples locations are shown in Appendix A.

Prior to drilling the borings, the field exploration locations were marked in the field and Underground Service Alert (USA) was notified.

Borings (DYB19-01, 02, 03, 04, 07, 08, 09, 10, 11 and 12) were drilled by ABC Liovin Drilling on April 25 and 26, 2019, with a truck-mounted CME-85 drill rig using hollow-stem auger drilling techniques. In addition, borings (DYB19-05,06,13,14) were performed on May 3, 2019 by Strongarm Environmental Field Services Inc. with a Geoprobe 5400 direct push sampler. Our field engineer observed the drilling operations and collected drive samples for visual examination and subsequent laboratory testing. Drive samples were collected with a 2.4-inch-inside-diameter (3.0-inch-outside-diameter) modified California split-barrel sampler lined with brass tubes and a standard split-spoon penetrometer with dimensions in accordance with ASTM D3550 and D1586, respectively. Both samplers were driven with a 140-pound automatic trip hammer falling 30 inches. Based on communication with the drilling company, the hammer used during the field exploration was last calculated on May 18, 2018; the efficiency rating (ER) was 79%.

The hammer blows required to drive the modified California sampler were converted to equivalent standard penetration test (SPT) N-values by multiplying by 0.65 (N = 0.65 x modified California blows per foot). A sampler driving refusal criteria of 50 hammer blows for less than 6 inches of penetration for the modified California or SPT samplers was used. An equivalent SPT blow count was then calculated by multiplying the sampler blow count (usually 50 blows) by the ratio of 6 inches divided by the actual sampler penetration in inches. If the modified California sampler met driving refusal, then the prorated equivalent SPT blow count was further modified as noted above for samples that did not meet sampler driving refusal.

Field unconfined compression strengths were obtained using a pocket penetrometer.

Soils encountered in the borings were classified in general accordance with the ASTM International (ASTM D2487, which is summarized on Plate B1, and D2488). Boring logs presented on Plates B2 through B18 were prepared from visual examination of the samples, cuttings obtained during drilling operations, and results of laboratory tests.

Groundwater was not encountered during the field exploration to a depth of 41.5 feet below the ground surface. Borings were backfilled with soil cuttings.

The boring locations were identified in the field by measuring from known locations using a hand-held global positioning system (GPS) unit.

SOIL CLASSIFICATION SYSTEM-ASTM D2487

MAJOR DIVISIONS			SYMBOLS: GRAPH	SYMBOLS: LETTER	TYPICAL DESCRIPTIONS
COARSE-GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE-GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

"Push" Sampler

Split Barrel "Drive" Sampler With Liner

Standard Penetration Test (SPT) Sampler

Dual-Mass Dynamic Cone Penetration (DCP) Test

Concrete/Rock Core

Groundwater Surface

SPT "N" = 0.65 x modified California blows per foot

NP = Nonplastic
EI = Expansion Index Test
SG = Specific Gravity
SE = Sand Equivalent
UC = Unconfined Comp.
CD = Consol. Drained Triaxial.
CU = Consol. Undrained Triaxial.
UU = Undrained, Unconsol. Triaxial.
RV = R-Value
CA = Chemical Analysis
DS = Direct Shear
CN = Consolidation
CP = Collapse Potential
SA = Grain size; HD = Hydrometer
MD = Compaction Test
HC = Hydraulic Conductivity Test
CBR = California Bearing Ratio
[PID] Reading in ppm above background

PLATE
B1

BORING LOCATION:	See Appendix A	**ELEVATION (feet):**	1549 NAVD88
LATITUDE:	33.86557	**LONGITUDE:**	-117.03065
DRILLING EQUIPMENT:	CME-85	**DRILLING METHOD:**	HSA
BORING DIAMETER (inches):	8	**BORING DEPTH (feet):**	6.5
DATE STARTED: 4-26-19	**COMPLETED:** 4-26-19	**HAMMER TYPE:** Automatic	**EFFICIENCY:** 79%
DRILLING CONTRACTOR:	ABC Liovin Drilling, Inc.	**HAMMER DROP:** 30 inches	**WEIGHT:** 140 lbs
LOGGED BY: BH	**CHECKED BY:** AA	**DRIVE SAMPLER DIAMETER (inches)**	**ID:** 2.4 **OD:** 3

Elevation (feet)	Depth (feet)	Sampler	Symbol	Blows per 6 Inches	SPT N60 Blows per Foot	Field Unc. Comp. Str. (tsf)	DESCRIPTION	Dry Density (pcf)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Percent Passing #200 Sieve	Other Tests [PID]
							SILTY SAND (SM): dark brown; moist; loose; medium to fine SAND; trace fine GRAVEL						SE
				6 3 2	7		micaceous					33	
1545	5			6 6 10	13		medium dense; coarse to fine SAND	111	5				
							Bottom of boring at 6.5 feet. Groundwater not encountered. Boring backfilled with soil cuttings.						
1540	10												
1535	15												
1530	20												
1525	25												
1520													

LOG OF BORING DYB19-01

Page 1 of 1

Gilman Springs RD
Project No. 2018-019

PLATE
B2

BORING LOCATION: See Appendix A		**ELEVATION (feet):** 1519 NAVD88	
LATITUDE: 33.86666		**LONGITUDE:** -117.03577	
DRILLING EQUIPMENT: CME-85		**DRILLING METHOD:** HSA	
BORING DIAMETER (inches): 8		**BORING DEPTH (feet):** 6.5	
DATE STARTED: 4-26-19	**COMPLETED:** 4-26-19	**HAMMER TYPE:** Automatic	**EFFICIENCY:** 79%
DRILLING CONTRACTOR: ABC Liovin Drilling, Inc.		**HAMMER DROP:** 30 inches	**WEIGHT:** 140 lbs
LOGGED BY: BH	**CHECKED BY:** AA	**DRIVE SAMPLER DIAMETER (inches)** **ID:** 2.4	**OD:** 3

Elevation (feet)	Depth (feet)	Sampler	Symbol	Blows per 6 Inches	SPT N60 Blows per Foot	Field Unc. Comp. Str. (tsf)	DESCRIPTION	Dry Density (pcf)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Percent Passing #200 Sieve	Other Tests [PID]
							SANDY LEAN CLAY (CL): brown; moist; very stiff; medium plasticity; medium to fine SAND; trace fine GRAVEL			33	21		MD RV
				17 16 17	28			111	8			53	
1515													
	5			3 6 8	18		fine SAND; no GRAVEL						
							Bottom of boring at 6.5 feet. Groundwater not encountered. Boring backfilled with soil cuttings.						
1510													
	10												
1505													
	15												
1500													
	20												
1495													
	25												
1490													

LOG OF BORING DYB19-02

Page 1 of 1

Gilman Springs RD
Project No. 2018-019

PLATE
B3

BORING LOCATION: See Appendix A		**ELEVATION (feet):** 1478 NAVD88	
LATITUDE: 33.8688		**LONGITUDE:** -117.04185	
DRILLING EQUIPMENT: CME-85		**DRILLING METHOD:** HSA	
BORING DIAMETER (inches): 8		**BORING DEPTH (feet):** 6.5	
DATE STARTED: 4-26-19	**COMPLETED:** 4-26-19	**HAMMER TYPE:** Automatic	**EFFICIENCY:** 79%
DRILLING CONTRACTOR: ABC Liovin Drilling, Inc.		**HAMMER DROP:** 30 inches	**WEIGHT:** 140 lbs
LOGGED BY: BH	**CHECKED BY:** AA	**DRIVE SAMPLER DIAMETER (inches)** **ID:** 2.4 **OD:** 3	

Elevation (feet)	Depth (feet)	Sampler	Symbol	Blows per 6 Inches	SPT N60 Blows per Foot	Field Unc. Comp. Str. (tsf)	DESCRIPTION	Dry Density (pcf)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Percent Passing #200 Sieve	Other Tests [PID]
							SILTY SAND (SM): dark olive brown; dry; loose; fine SAND; trace fine GRAVEL; micaceous					31	CA EI MD RV
1475				4 3 4	9								
	5			9 7 5	11		olive brown; medium dense	112	5				
							Bottom of boring at 6.5 feet. Groundwater not encountered. Boring backfilled with Soil Cuttings.						
1470													
	10												
1465													
	15												
1460													
	20												
1455													
	25												
1450													

LOG OF BORING DYB19-03

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PLATE
B4

BORING LOCATION: See Appendix A		**ELEVATION (feet):** 1448 NAVD88	
LATITUDE: 33.87241		**LONGITUDE:** -117.04822	
DRILLING EQUIPMENT: Hand Auger		**DRILLING METHOD:** Hand Auger	
BORING DIAMETER (inches): 3.25		**BORING DEPTH (feet):** 0.83	
DATE STARTED: 4-26-19	**COMPLETED:** 4-26-19	**HAMMER TYPE:** N/A	**EFFICIENCY:** N/A%
DRILLING CONTRACTOR: ABC Liovin Drilling, Inc.		**HAMMER DROP:** N/A inches	**WEIGHT:** N/A lbs
LOGGED BY: BH	**CHECKED BY:** AA	**DRIVE SAMPLER DIAMETER (inches)**	**ID:** 2.4 **OD:** 3

Elevation (feet)	Depth (feet)	Sampler	Symbol	Blows per 6 Inches	SPT N60 Blows per Foot	Field Unc. Comp. Str. (tsf)	DESCRIPTION	Dry Density (pcf)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Percent Passing #200 Sieve	Other Tests [PID]
			SM				SILTY SAND (SM): brown; dry; very dense; fine SAND; micaceous						
							Bottom of boring at 10 inches. Refusal encountered due to very dense material						
1445													
	5												
1440													
	10												
1435													
	15												
1430													
	20												
1425													
	25												
1420													

LOG OF BORING DYB19-04

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PLATE
B5

BORING LOCATION: See Appendix A		**ELEVATION (feet):** 1435 NAVD88	
LATITUDE: 33.87598		**LONGITUDE:** -117.05212	
DRILLING EQUIPMENT: Geoprobe 5400		**DRILLING METHOD:** Direct Push	
BORING DIAMETER (inches): 2.25		**BORING DEPTH (feet):** 12	
DATE STARTED: 5-3-19	**COMPLETED:** 5-3-19	**HAMMER TYPE:** N/A	**EFFICIENCY:** N/A%
DRILLING CONTRACTOR: Strongarm Environmental Field		**HAMMER DROP:** N/A inches	**WEIGHT:** N/A lbs
LOGGED BY: BH	**CHECKED BY:** SN	**DRIVE SAMPLER DIAMETER (inches)** **ID:** 2.4 **OD:** 3	

Elevation (feet)	Depth (feet)	Sampler	Symbol	Blows per 6 Inches	SPT N60 Blows per Foot	Field Unc. Comp. Str. (tsf)	DESCRIPTION	Dry Density (pcf)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Percent Passing #200 Sieve	Other Tests [PID]
							SILTY SAND (SM): brown; moist; medium dense; coarse to fine SAND; trace fine GRAVEL; FILL: trace asphalt						
							SANDY LEAN CLAY (CL): brown; moist; medium plasticity; medium to fine SAND; trace fine GRAVEL	110	14	30	16	50	
1430	5												
							medium stiff; fine SAND; no trace fine GRAVEL						
						0.75		112	18	30	17	63	
1425	10												
							Bottom of boring at 12 feet. Groundwater not encountered. Boring backfilled with granular bentonite.						
1420	15												
1415	20												
1410	25												

LOG OF BORING DYB19-05

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PLATE
B6

BORING LOCATION: See Appendix A		**ELEVATION (feet):** 1437 NAVD88	
LATITUDE: 33.87972		**LONGITUDE:** -117.05685	
DRILLING EQUIPMENT: Geoprobe 5400		**DRILLING METHOD:** Direct Push	
BORING DIAMETER (inches): 2.25		**BORING DEPTH (feet):** 12	
DATE STARTED: 5-3-19	**COMPLETED:** 5-3-19	**HAMMER TYPE:** N/A	**EFFICIENCY:** N/A%
DRILLING CONTRACTOR: Strongarm Environmental Field		**HAMMER DROP:** N/A inches	**WEIGHT:** N/A lbs
LOGGED BY: BH	**CHECKED BY:** SN	**DRIVE SAMPLER DIAMETER (inches)**	**ID:** 2.4 **OD:** 3

Elevation (feet)	Depth (feet)	Sampler	Symbol	Blows per 6 Inches	SPT N60 Blows per Foot	Field Unc. Comp. Str. (tsf)	DESCRIPTION	Dry Density (pcf)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Percent Passing #200 Sieve	Other Tests [PID]
1435							CLAYEY SAND (SC): brown; moist; medium dense; medium to fine SAND						
										30	14	38	CA
	5						fine SAND; increased fines; trace CLAY						
1430								102	9				
	10						SANDY SILTY CLAY (CL-ML): brown; moist; fine SAND					54	
1425							Bottom of boring at 12 feet. Groundwater not encountered. Boring backfilled with granular bentonite.						
	15												
1420													
	20												
1415													
	25												
1410													

LOG OF BORING DYB19-06

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PLATE
B7

BORING LOCATION: See Appendix A		**ELEVATION (feet):** 1480 NAVD88	
LATITUDE: 33.88526		**LONGITUDE:** -117.06348	
DRILLING EQUIPMENT: CME-85		**DRILLING METHOD:** HSA	
BORING DIAMETER (inches): 8		**BORING DEPTH (feet):** 6.5	
DATE STARTED: 4-26-19	**COMPLETED:** 4-26-19	**HAMMER TYPE:** Automatic	**EFFICIENCY:** 79%
DRILLING CONTRACTOR: ABC Liovin Drilling, Inc.		**HAMMER DROP:** 30 inches	**WEIGHT:** 140 lbs
LOGGED BY: BH	**CHECKED BY:** AA	**DRIVE SAMPLER DIAMETER (inches)** **ID:** 2.4 **OD:** 3	

Elevation (feet)	Depth (feet)	Sampler	Symbol	Blows per 6 Inches	SPT N60 Blows per Foot	Field Unc. Comp. Str. (tsf)	DESCRIPTION	Dry Density (pcf)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Percent Passing #200 Sieve	Other Tests [PID]
				3 2 3	4		SILTY SAND with GRAVEL (SM): brown; moist; very loose; fine SAND; coarse to fine GRAVEL; micaceous	99	5				MD RV SE
1475	5			3 4 4	11		POORLY GRADED SAND with SILT and GRAVEL (SP-SM): light yellowish brown; moist; medium dense; coarse to fine SAND; coarse to fine GRAVEL; micaceous					6	
							Bottom of boring at 6.5 feet. Groundwater not encountered. Boring backfilled with soil cuttings.						
1470	10												
1465	15												
1460	20												
1455	25												

LOG OF BORING DYB19-07

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PLATE
B8

BORING LOCATION: See Appendix A		**ELEVATION (feet):** 1514 NAVD88	
LATITUDE: 33.88976		**LONGITUDE:** -117.07087	
DRILLING EQUIPMENT: CME-85		**DRILLING METHOD:** HSA	
BORING DIAMETER (inches): 8		**BORING DEPTH (feet):** 6.5	
DATE STARTED: 4-25-19	**COMPLETED:** 4-25-19	**HAMMER TYPE:** Automatic	**EFFICIENCY:** 79%
DRILLING CONTRACTOR: ABC Liovin Drilling, Inc.		**HAMMER DROP:** 30 inches	**WEIGHT:** 140 lbs
LOGGED BY: BH	**CHECKED BY:** AA	**DRIVE SAMPLER DIAMETER (inches)**	**ID:** 2.4 **OD:** 3

Elevation (feet)	Depth (feet)	Sampler	Symbol	Blows per 6 Inches	SPT N60 Blows per Foot	Field Unc. Comp. Str. (tsf)	DESCRIPTION	Dry Density (pcf)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Percent Passing #200 Sieve	Other Tests [PID]
							SILTY SAND with GRAVEL (SM): brown; dry; medium dense; medium to fine SAND; coarse to fine GRAVEL						SE
				6 10 15	21		POORLY GRADED SAND with SILT and GRAVEL (SP-SM): light yellowish brown; dry; medium dense; medium to fine SAND; coarse to fine GRAVEL	110	3				
1510	5			6 9 8	22		moist					9	
							Bottom of boring at 6.5 feet. Groundwater not encountered. Boring backfilled with soil cuttings.						
1505	10												
1500	15												
1495	20												
1490	25												
1485													

LOG OF BORING DYB19-08

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PLATE
B9

BORING LOCATION: See Appendix A		**ELEVATION (feet):** 1510 NAVD88	
LATITUDE: 33.88977		**LONGITUDE:** -117.07195	
DRILLING EQUIPMENT: CME-85		**DRILLING METHOD:** HSA	
BORING DIAMETER (inches): 8		**BORING DEPTH (feet):** 41.5	
DATE STARTED: 4-25-19	**COMPLETED:** 4-25-19	**HAMMER TYPE:** Automatic	**EFFICIENCY:** 79%
DRILLING CONTRACTOR: ABC Liovin Drilling, Inc.		**HAMMER DROP:** 30 inches	**WEIGHT:** 140 lbs
LOGGED BY: BH	**CHECKED BY:** AA	**DRIVE SAMPLER DIAMETER (inches)**	**ID:** 2.4 **OD:** 3

Elevation (feet)	Depth (feet)	Sampler	Symbol	Blows per 6 Inches	SPT N60 Blows per Foot	Field Unc. Comp. Str. (tsf)	DESCRIPTION	Dry Density (pcf)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Percent Passing #200 Sieve	Other Tests [PID]
							ASPHALT CONCRETE (AC): 10-inches						
							POORLY GRADED GRAVEL with SILT and SAND (GP-GM): dark brown/black; moist; medium dense; medium to fine SAND; coarse to fine GRAVEL; Base: 3-inches						
				8 10 9	25		SILTY SAND (SM): brown; moist; medium dense; medium to fine SAND; trace fine GRAVEL; micaceous						
1505	5			4 13 2	13		SILTY SAND with GRAVEL (SM): brown; moist; medium dense; medium to fine SAND; coarse to fine GRAVEL; trace CLAY; micaceous	99	7				CA
1500	10			11 15 17	42		dense; no recovery; coarse GRAVEL stuck in sampler shoe						
							SILTY SAND (SM): light olive brown; moist; medium dense; fine SAND						
1495	15			5 6 7	11			104	5			30	DS
1490	20			7 18 35	70		very dense; medium to fine SAND; trace coarse to fine GRAVEL; decrease fines						
1485	25			5 15 14	25		medium dense; increased fines; trace CLAY	108	7	NP	NP	30	
							POORLY GRADED SAND with SILT and GRAVEL (SP-SM): white/tan; moist; very dense; coarse to fine SAND; coarse to fine GRAVEL; oxidation stains						

LOG OF BORING DYB19-09

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PLATE
B10

Elevation (feet)	Depth (feet)	Sampler	Symbol	Blows per 6 Inches	SPT N60 Blows per Foot	Field Unc. Comp. Str. (tsf)	DESCRIPTION	Dry Density (pcf)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Percent Passing #200 Sieve	Other Tests [PID]
				17 28 29	75								
1475	35			18 22 20	36		dense; no oxidation	107	4				
1470	40			19 34 37	93		very dense; increased fines						
							Bottom of boring at 41.5 feet. Groundwater not encountered. Boring backfilled with soil cuttings. Surface patched with cold patch asphalt.						
1465	45												
1460	50												
1455	55												
1450	60												
1445	65												

LOG OF BORING DYB19-09

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PLATE

B11

BORING LOCATION: See Appendix A		**ELEVATION (feet):** 1505 NAVD88	
LATITUDE: 33.88993		**LONGITUDE:** -117.07303	
DRILLING EQUIPMENT: CME-85		**DRILLING METHOD:** HSA	
BORING DIAMETER (inches): 8		**BORING DEPTH (feet):** 31.5	
DATE STARTED: 4-25-19	**COMPLETED:** 4-25-19	**HAMMER TYPE:** Automatic	**EFFICIENCY:** 79%
DRILLING CONTRACTOR: ABC Liovin Drilling, Inc.		**HAMMER DROP:** 30 inches	**WEIGHT:** 140 lbs
LOGGED BY: BH	**CHECKED BY:** AA	**DRIVE SAMPLER DIAMETER (inches)** **ID:** 2.4	**OD:** 3

Elevation (feet)	Depth (feet)	Sampler	Symbol	Blows per 6 Inches	SPT N60 Blows per Foot	Field Unc. Comp. Str. (tsf)	DESCRIPTION	Dry Density (pcf)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Percent Passing #200 Sieve	Other Tests [PID]
							ASPHALT CONCRETE (AC): 7-inches						
							SILTY SAND with GRAVEL (SM): dark brown; moist; medium dense; medium to fine SAND; coarse to fine GRAVEL; Base: 8-inches						
				15 13 13	22		SILTY SAND (SM): brown; moist; medium dense; medium to fine SAND; trace fine GRAVEL; trace CLAY; micaceous	110	9				
1500	5			1 1 3	5		olive brown; loose; fine SAND; increased fines; no trace fine GRAVEL					33	CA
							SILTY SAND with GRAVEL (SM): light gray; dry; medium dense; medium to fine SAND; coarse to fine GRAVEL						
1495	10			13 13 15	24			114	1				DS
							POORLY GRADED SAND with SILT and GRAVEL (SP-SM): light gray; dry; medium dense; coarse to fine SAND; coarse to fine GRAVEL						
				10 14 15	25			117	1				
1490	15												
							SILTY SAND (SM): brown; dry; medium dense; fine SAND						
1485	20			10 13 14	24			104	3			30	DS
1480	25												
							POORLY GRADED SAND with SILT and GRAVEL (SP-SM): brown; dry; very dense; coarse to fine SAND; coarse to fine GRAVEL						

LOG OF BORING DYB19-10

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PLATE
B12

Elevation (feet)	Depth (feet)	Sampler	Symbol	Blows per 6 Inches	SPT N60 Blows per Foot	Field Unc. Comp. Str. (tsf)	DESCRIPTION	Dry Density (pcf)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Percent Passing #200 Sieve	Other Tests [PID]
				13 28 50/6"	71			119	2				
							Bottom of boring at 31.5 feet. Groundwater not encountered. Boring backfilled with soil cuttings.						
1470	35												
1465	40												
1460	45												
1455	50												
1450	55												
1445	60												
1440	65												

LOG OF BORING DYB19-10

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PLATE
B13

BORING LOCATION: See Appendix A		**ELEVATION (feet):** 1494 NAVD88	
LATITUDE: 33.8901		**LONGITUDE:** -117.07422	
DRILLING EQUIPMENT: CME-85		**DRILLING METHOD:** HSA	
BORING DIAMETER (inches): 8		**BORING DEPTH (feet):** 38.5	
DATE STARTED: 4-25-19	**COMPLETED:** 4-25-19	**HAMMER TYPE:** Automatic	**EFFICIENCY:** 79%
DRILLING CONTRACTOR: ABC Liovin Drilling, Inc.		**HAMMER DROP:** 30 inches	**WEIGHT:** 140 lbs
LOGGED BY: BH	**CHECKED BY:** AA	**DRIVE SAMPLER DIAMETER (inches)** **ID:** 2.4	**OD:** 3

Elevation (feet)	Depth (feet)	Sampler	Symbol	Blows per 6 Inches	SPT N60 Blows per Foot	Field Unc. Comp. Str. (tsf)	DESCRIPTION	Dry Density (pcf)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Percent Passing #200 Sieve	Other Tests [PID]
							ASPHALT CONCRETE (AC): 4-inches						
							POORLY GRADED GRAVEL with SILT and SAND (GP-GM): dark brown; moist; medium dense; coarse to fine SAND; coarse to fine GRAVEL; Base:7-inches						
				50 32 32	55		SILTY SAND with GRAVEL (SM): brown; dry; very dense; coarse to fine SAND; coarse to fine GRAVEL; micaceous	116	3				
1490													
	5			4 3 4	9		olive brown; loose; fine SAND; coarse to fine GRAVEL; increased fines						CA
							SILTY SAND (SM): brown; dry; medium dense; fine SAND; trace coarse to fine GRAVEL						
1485													
	10			4 6 10	13			100	3			19	DS
1480													
	15												
				8 8 11	25								
1475													
	20												
				13 20 26	40		dense; increased coarse GRAVEL	113	1			17	DS
1470													
	25												
				7 9 9	24		medium dense; trace coarse to fine GRAVEL						
1465													

LOG OF BORING DYB19-11

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PLATE
B14

Elevation (feet)	Depth (feet)	Sampler	Symbol	Blows per 6 Inches	SPT N60 Blows per Foot	Field Unc. Comp. Str. (tsf)	DESCRIPTION	Dry Density (pcf)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Percent Passing #200 Sieve	Other Tests [PID]
				12 16 18	29		trace fine GRAVEL; increased fines	101	3				
1460	35												
				16 40 50	100		very dense; increased coarse SAND; trace fine GRAVEL						
1455	40						Bottom of boring at 38.5 feet. Groundwater not encountered. Boring backfilled with soil cuttings. Surface patched with cold patch asphalt.						
1450	45												
1445	50												
1440	55												
1435	60												
1430	65												
1425													

LOG OF BORING DYB19-11

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PLATE
B15

BORING LOCATION: See Appendix A		**ELEVATION (feet):** 1469 NAVD88	
LATITUDE: 33.89155		**LONGITUDE:** -117.07804	
DRILLING EQUIPMENT: CME-85		**DRILLING METHOD:** HSA	
BORING DIAMETER (inches): 8		**BORING DEPTH (feet):** 6.5	
DATE STARTED: 4-26-19	**COMPLETED:** 4-26-19	**HAMMER TYPE:** Automatic	**EFFICIENCY:** 79%
DRILLING CONTRACTOR: ABC Liovin Drilling, Inc.		**HAMMER DROP:** 30 inches	**WEIGHT:** 140 lbs
LOGGED BY: BH	**CHECKED BY:** AA	**DRIVE SAMPLER DIAMETER (inches)**	**ID:** 2.4 **OD:** 3

Elevation (feet)	Depth (feet)	Sampler	Symbol	Blows per 6 Inches	SPT N60 Blows per Foot	Field Unc. Comp. Str. (tsf)	DESCRIPTION	Dry Density (pcf)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Percent Passing #200 Sieve	Other Tests [PID]
							SILTY SAND (SM): brown; moist; medium dense; medium to fine SAND; trace fine GRAVEL						
				8 7 8	13			97	7			32	
1465	5			2 2 2	5		loose; coarse to fine SAND; trace coarse to fine GRAVEL; decrease fines						
							Bottom of boring at 6.5 feet. Groundwater not encountered. Boring backfilled with soil cuttings.						
1460	10												
1455	15												
1450	20												
1445	25												
1440													

LOG OF BORING DYB19-12

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PLATE
B16

BORING LOCATION:	See Appendix A	**ELEVATION (feet):**	1453 NAVD88
LATITUDE:	33.89627	**LONGITUDE:**	-117.08197
DRILLING EQUIPMENT:	Geoprobe 5400	**DRILLING METHOD:**	Direct Push
BORING DIAMETER (inches):	2.25	**BORING DEPTH (feet):**	12
DATE STARTED: 5-3-19	**COMPLETED:** 5-3-19	**HAMMER TYPE:** N/A	**EFFICIENCY:** N/A%
DRILLING CONTRACTOR:	Strongarm Environmental Field	**HAMMER DROP:** N/A inches	**WEIGHT:** N/A lbs
LOGGED BY: BH	**CHECKED BY:** SN	**DRIVE SAMPLER DIAMETER (inches)**	**ID:** 2.4 **OD:** 3

Elevation (feet)	Depth (feet)	Sampler	Symbol	Blows per 6 Inches	SPT N60 Blows per Foot	Field Unc. Comp. Str. (tsf)	DESCRIPTION	Dry Density (pcf)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Percent Passing #200 Sieve	Other Tests [PID]
							SILTY SAND (SM): brown; moist; medium dense; medium to fine SAND; trace coarse to fine GRAVEL			NP	NP	40	CA EI MD RV SE
1450													
							SANDY LEAN CLAY (CL): brown; moist; medium plasticity; fine SAND						
	5									38	25	65	
							LEAN CLAY with SAND (CL): brown; moist; medium plasticity; fine SAND						
1445													
	10									39	26	72	
							dark gray						
							Bottom of boring at 12 feet. Groundwater not encountered. Boring backfilled with granular bentonite.						
1440													
	15												
1435													
	20												
1430													
	25												
1425													

LOG OF BORING DYB19-13

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PLATE

B17

BORING LOCATION: See Appendix A		**ELEVATION (feet):** 1444 NAVD88	
LATITUDE: 33.90050		**LONGITUDE:** -117.08531	
DRILLING EQUIPMENT: Geoprobe 5400		**DRILLING METHOD:** Direct Push	
BORING DIAMETER (inches): 2.25		**BORING DEPTH (feet):** 12	
DATE STARTED: 5-3-19	**COMPLETED:** 5-3-19	**HAMMER TYPE:** N/A	**EFFICIENCY:** N/A%
DRILLING CONTRACTOR: Strongarm Environmental Field		**HAMMER DROP:** N/A inches	**WEIGHT:** N/A lbs
LOGGED BY: BH	**CHECKED BY:** SN	**DRIVE SAMPLER DIAMETER (inches)**	**ID:** 2.4 **OD:** 3

Elevation (feet)	Depth (feet)	Sampler	Symbol	Blows per 6 Inches	SPT N60 Blows per Foot	Field Unc. Comp. Str. (tsf)	DESCRIPTION	Dry Density (pcf)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Percent Passing #200 Sieve	Other Tests [PID]
							SILTY SAND (SM): brown; moist; fine SAND; trace rootlet						
							medium to fine SAND					43	
1440	5						fine SAND						
								108	5				
1435	10						trace coarse GRAVEL						
								112	10				
							Bottom of boring at 12 feet. Groundwater not encountered. Boring backfilled with granular bentonite.						
1430	15												
1425	20												
1420	25												
1415													

LOG OF BORING DYB19-14

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PLATE
B18

APPENDIX C -
LABORATORY TESTING

APPENDIX C - LABORATORY TESTING

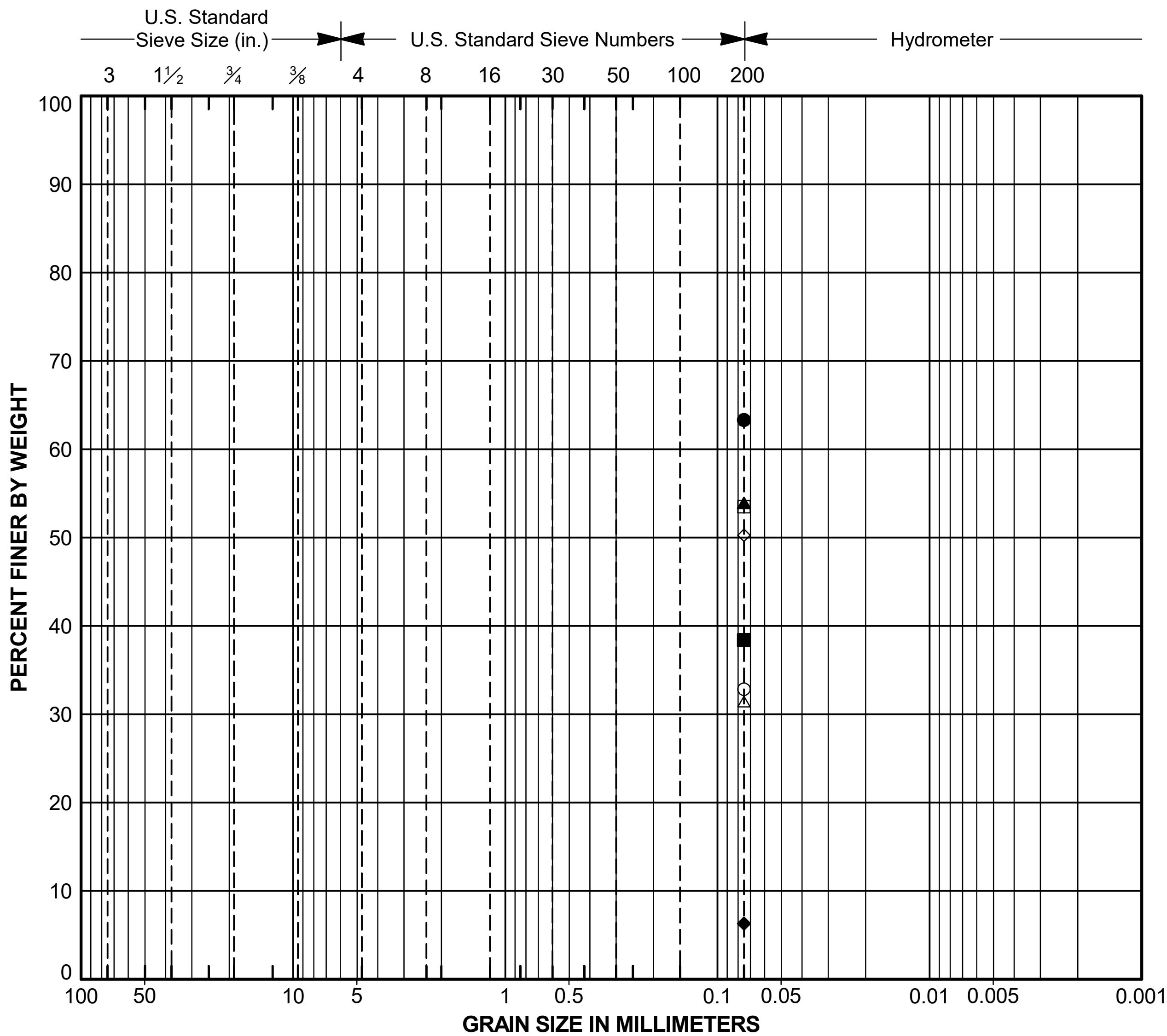
Diaz•Yourman & Associates (DYA) selected soil samples to be tested and the tests to be performed on the selected samples. Laboratory testing was performed by Hushmand Associates, Inc. Laboratory data are summarized on the boring logs in Appendix B and presented on Plates C1 through C29. A summary of the geotechnical laboratory testing is presented in Table C1.

Table C1 - LABORATORY TESTING SUMMARY

TEST NAME	PROCEDURE	PURPOSE	LOCATION
Percent Passing the No. 200 Sieve	ASTM D1140	Classification, index properties	Boring Logs
Moisture Content, Dry Density	ASTM D2216	Classification, index properties	Boring Logs
Grain-Size Distribution	ASTM D422	Classification, index properties	Plates C1 – C3
Atterberg Limits	ASTM D4318	Expansion potential, classification, index properties	Plate C4
Direct Shear	ASTM D3080	Shear strength	Plates C15 - C19
Expansion Index	ASTM D4829	Expansion potential	Plate C9 and C10
Compaction	ASTM D1557	Earthwork	Plates C5 – C8
Sand Equivalent	CTM 217	Earthwork	Plates C11 – C14
Resistance (R-) Value	ASTM D2844 CTM 301	Pavement thickness design	Plates C20 – C28
pH	ASTM G51	Corrosion potential	Table C2 or Plate C29
Resistivity	ASTM G187	Corrosion potential	Table C2 or Plate C29
Soluble Sulfates	ASTM D4327	Corrosion potential	Table C2 or Plate C29
Soluble Chlorides	ASTM D4327	Corrosion potential	Table C2 or Plate C29

Note(s):

- ASTM = ASTM International
- CTM = Caltrans Test Method



COBBLES	GRAVEL		SAND			SILT or CLAY
	Coarse	Fine	Coarse	Medium	Fine	

Laboratory Testing by:

Symbol	Source	Depth (feet)	Classification	Natural M. C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
⊙	DYB19-01	2.5	SILTY SAND (SM)				33
□	DYB19-02	1.5	SANDY LEAN CLAY (CL)	8			53
△	DYB19-03	0.0	SILTY SAND (SM)				31
◇	DYB19-05	3.0	SANDY LEAN CLAY (CL)	14	30	16	50
●	DYB19-05	7.0	SANDY LEAN CLAY (CL)	18	30	17	63
■	DYB19-06	3.0	CLAYEY SAND (SC)		30	14	38
▲	DYB19-06	10.0	SANDY SILTY CLAY (CL-ML)				54
◆	DYB19-07	5.0	POORLY GRADED SAND WITH SILT AND GRAVEL				6

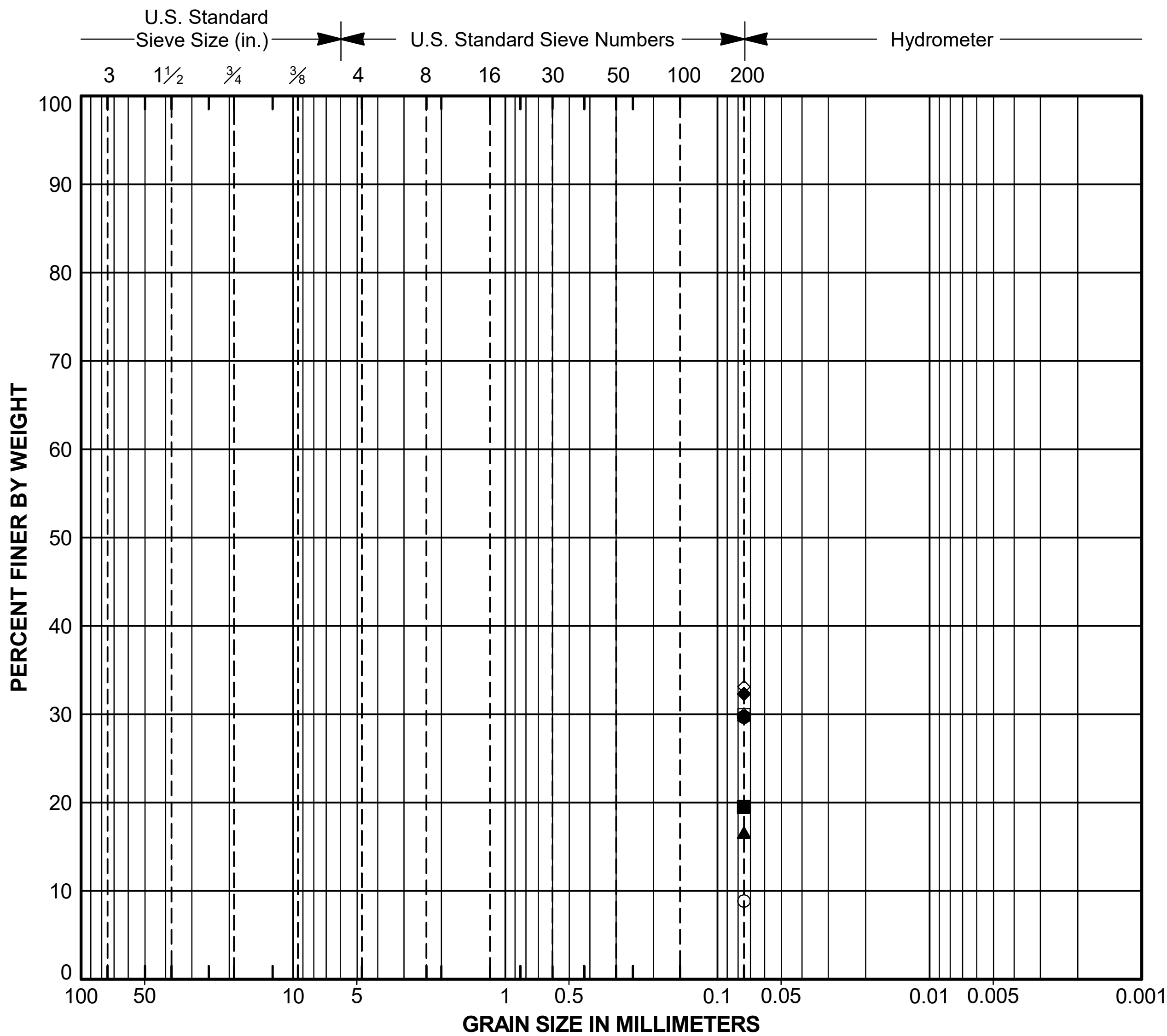
PARTICLE SIZE ANALYSIS

Gilman Springs RD

Project No. 2018-019

PLATE

C1



COBBLES	GRAVEL		SAND			SILT or CLAY
	Coarse	Fine	Coarse	Medium	Fine	

Laboratory Testing by:

Symbol	Source	Depth (feet)	Classification	Natural M. C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
○	DYB19-08	5.0	POORLY GRADED SAND WITH SILT AND GRAVEL				9
□	DYB19-09	15.0	SILTY SAND (SM)	5			30
△	DYB19-09	25.0	SILTY SAND (SM)	7	NP	NP	30
◇	DYB19-10	5.0	SILTY SAND (SM)				33
●	DYB19-10	20.0	SILTY SAND (SM)	3			30
■	DYB19-11	10.0	SILTY SAND (SM)	3			19
▲	DYB19-11	23.0	SILTY SAND (SM)	1			17
◆	DYB19-12	1.5	SILTY SAND (SM)	7			32

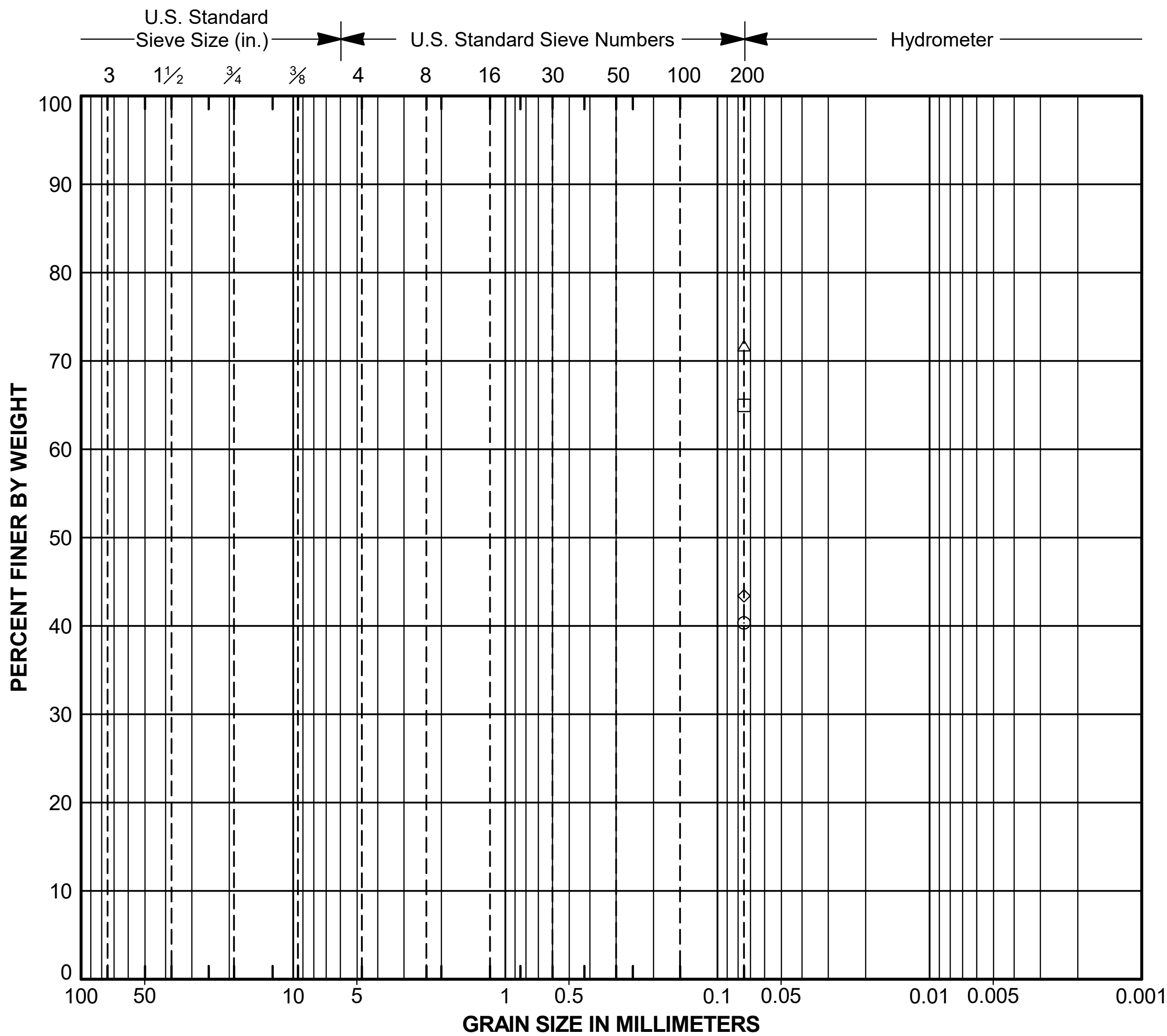
PARTICLE SIZE ANALYSIS

Gilman Springs RD

Project No. 2018-019

PLATE

C2



COBBLES	GRAVEL		SAND			SILT or CLAY
	Coarse	Fine	Coarse	Medium	Fine	

Laboratory Testing by:

Symbol	Source	Depth (feet)	Classification	Natural M. C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
○	DYB19-13	0.0	SILTY SAND (SM)		NP	NP	40
□	DYB19-13	5.0	SANDY LEAN CLAY (CL)		38	25	65
△	DYB19-13	9.0	LEAN CLAY WITH SAND (CL)		39	26	72
◇	DYB19-14	3.0	SILTY SAND (SM)				43

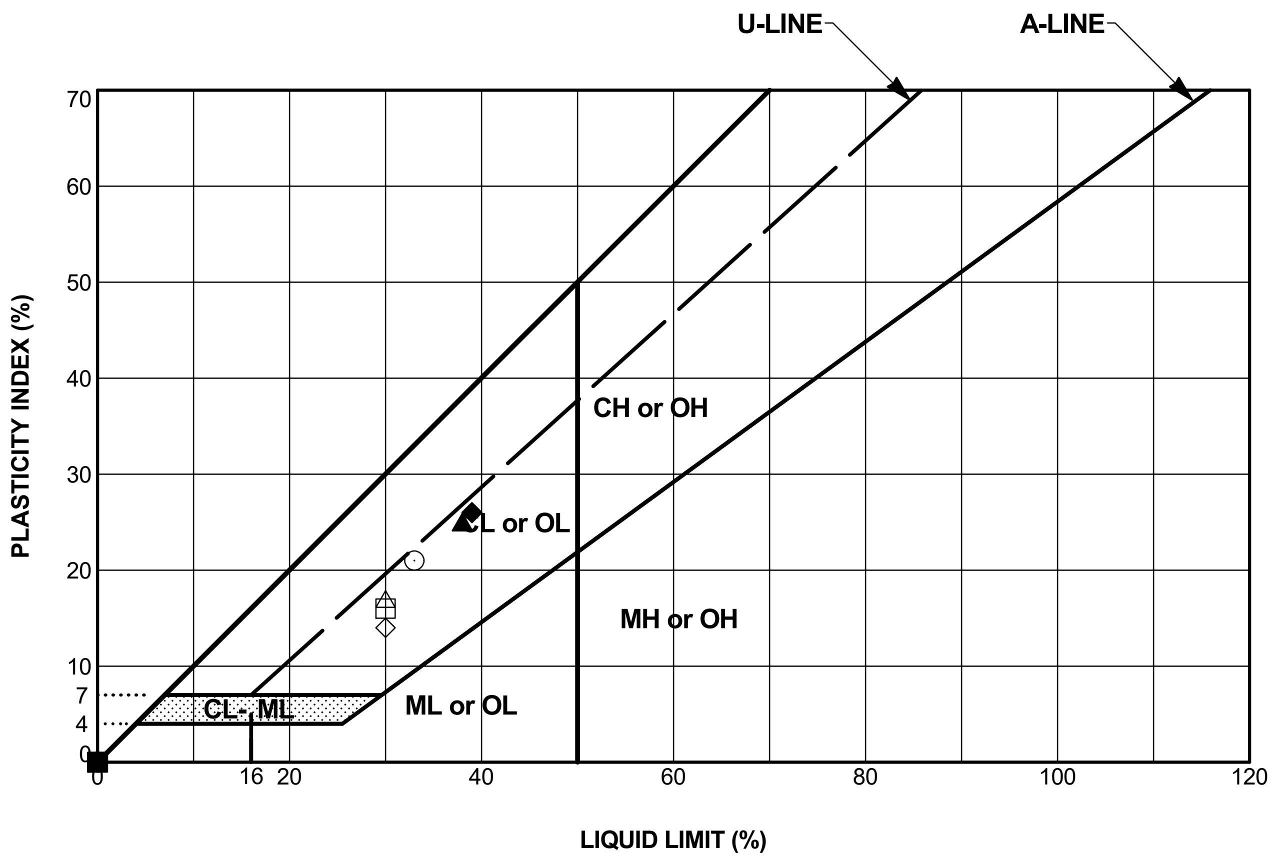
PARTICLE SIZE ANALYSIS

Gilman Springs RD

Project No. 2018-019

PLATE

C3



Laboratory Testing by:

Test Method: ASTM D4318

Symbol	Source	Depth (feet)	Classification	Natural M. C. (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
⊙	DYB19-02	0.0	SANDY LEAN CLAY (CL)		33	12	21	
□	DYB19-05	3.0	SANDY LEAN CLAY (CL)	14	30	14	16	50
△	DYB19-05	7.0	SANDY LEAN CLAY (CL)	18	30	13	17	63
◇	DYB19-06	3.0	CLAYEY SAND (SC)		30	16	14	38
●	DYB19-09	25.0	SILTY SAND (SM)	7	NP	NP	NP	30
■	DYB19-13	0.0	SILTY SAND (SM)		NP	NP	NP	40
▲	DYB19-13	5.0	SANDY LEAN CLAY (CL)		38	13	25	65
◆	DYB19-13	9.0	LEAN CLAY WITH SAND (CL)		39	13	26	72

PLASTICITY CHART

Gilman Springs RD

Project No. 2018-019

PLATE
C4

COMPACTION CHARACTERISTICS OF SOILS

ASTM D1557

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

Client :	Diaz Yourman & Associates	***HAI Project No.:***	DYAL-19-010
Project Name:	Gilman Springs Rd	***Tested by:***	GA
Project Number:	2018-019	***Checked by:***	KL
Boring Number:	DYB19-02	***Date:***	05/08/19
Sample No:	0 (Bulk)	***Mold size (in):***	6"
Depth (ft) :	0-5	***Procedure:***	C
Soil Description:	Brown, Clayey Sand with Gravel (SC)	***% Ret. on 3/4":***	3.7

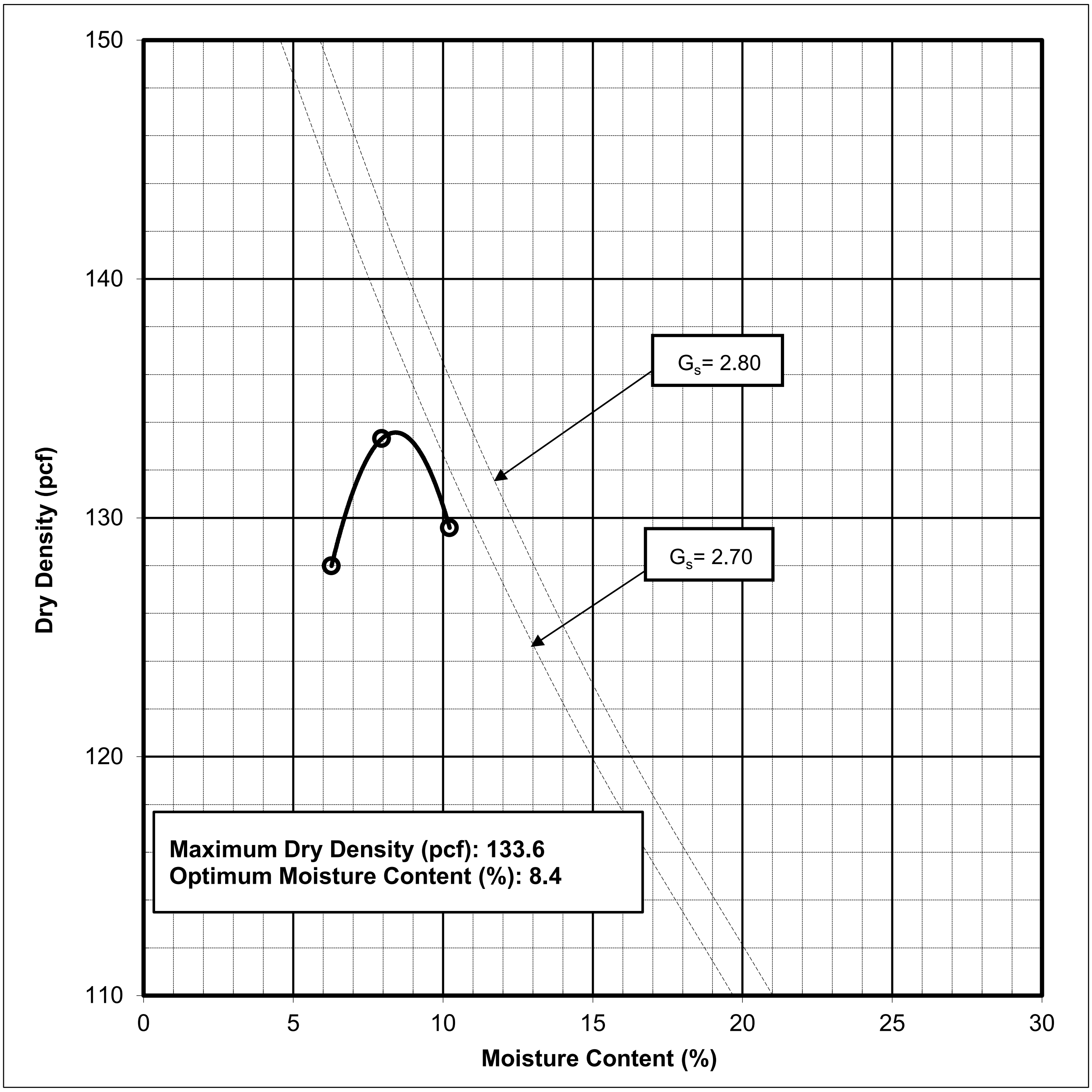
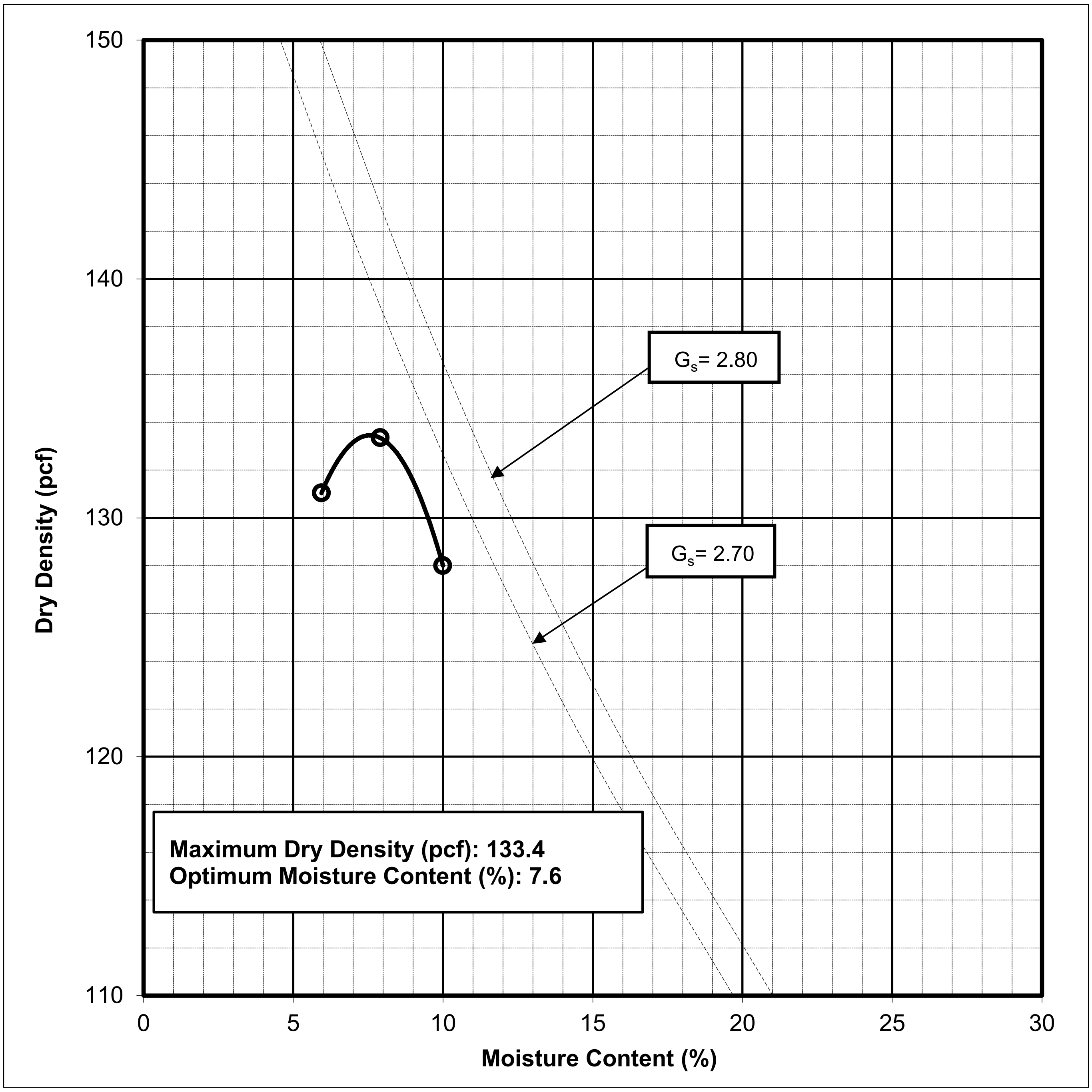


PLATE
C5

COMPACTION CHARACTERISTICS OF SOILS
ASTM D1557

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

Client :	Diaz Yourman & Associates	***HAI Project No.:***	DYAL-19-010
Project Name:	Gilman Springs Rd	***Tested by:***	GA
Project Number:	2018-019	***Checked by:***	KL
Boring Number:	DYB19-03	***Date:***	05/08/19
Sample No:	0 (Bulk)	***Mold size (in):***	6"
Depth (ft) :	0-5	***Procedure:***	C
Soil Description:	Brown, Silty Sand (SM)	***% Ret. on 3/4":***	2.1



Maximum Dry Density (pcf): 133.4
Optimum Moisture Content (%): 7.6

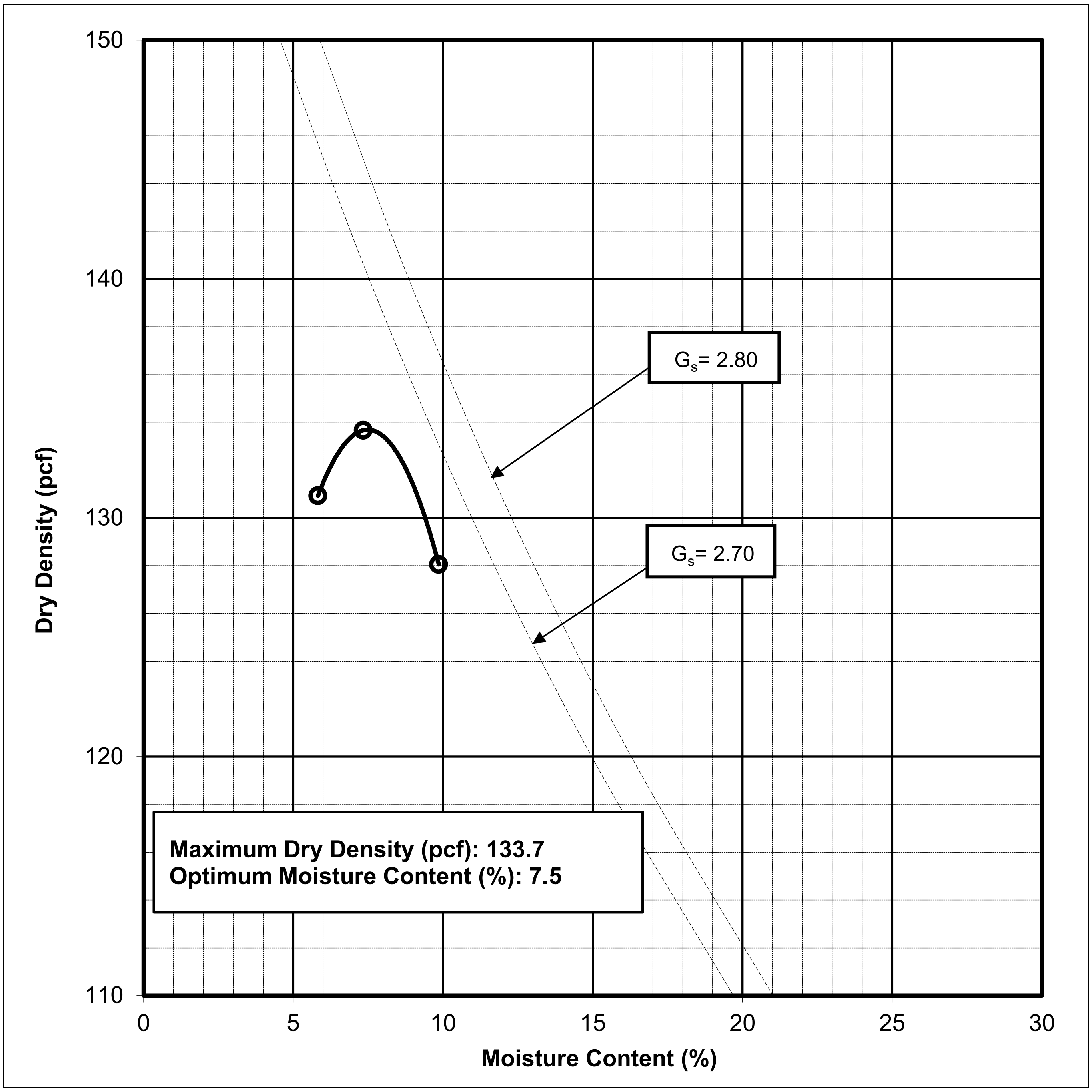
PLATE
C6

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

COMPACTION CHARACTERISTICS OF SOILS

ASTM D1557

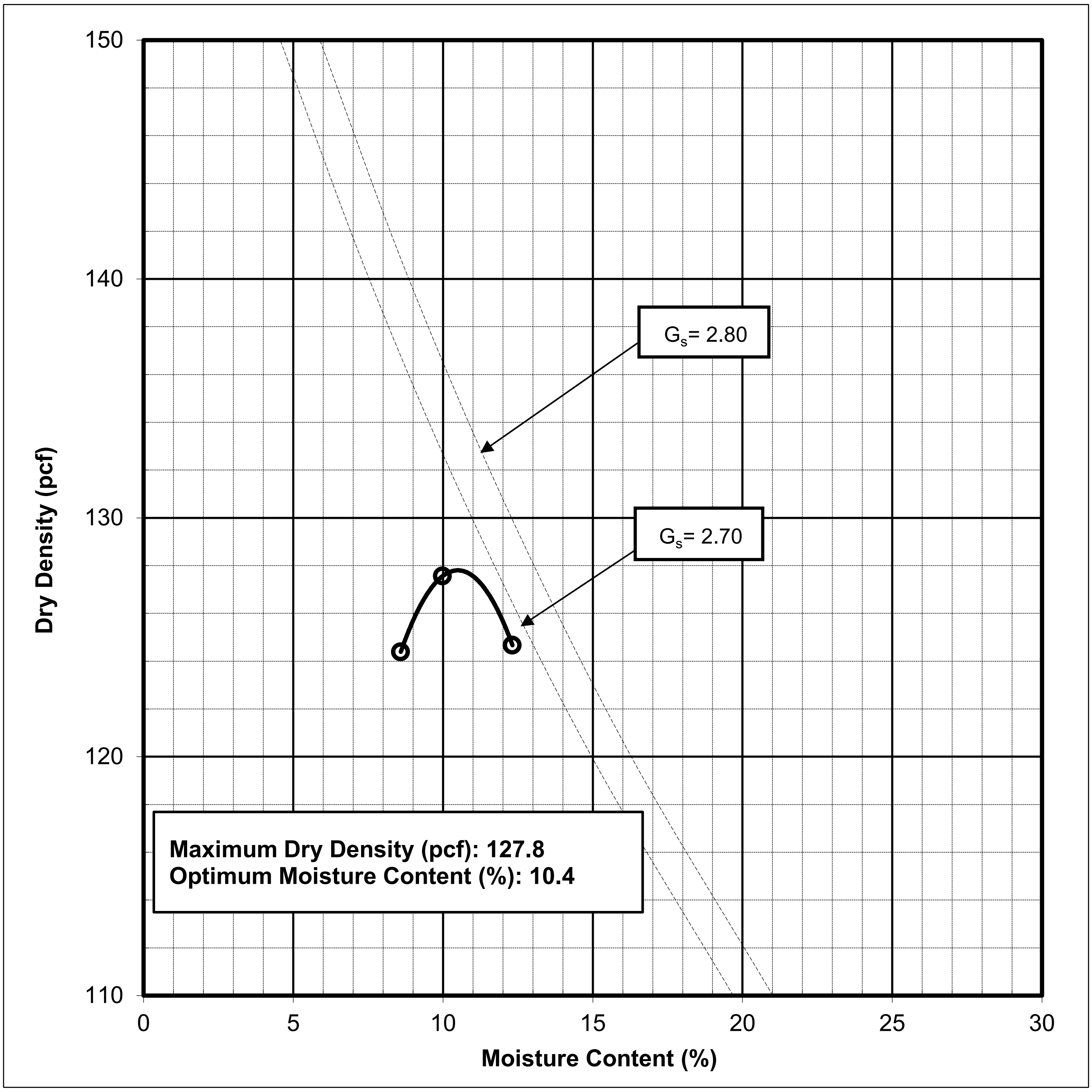
Client :	Diaz Yourman & Associates	***HAI Project No.:***	DYAL-19-010
Project Name:	Gilman Springs Rd	***Tested by:***	GA
Project Number:	2018-019	***Checked by:***	KL
Boring Number:	DYB19-07	***Date:***	05/08/19
Sample No:	0 (Bulk)	***Mold size (in):***	6"
Depth (ft) :	0-5	***Procedure:***	C
Soil Description:	Brown, Silty Sand with Gravel (SM)	***% Ret. on 3/4":***	3.0



COMPACTION CHARACTERISTICS OF SOILS

ASTM D1557

Client :	Diaz Yourman & Associates	***HAI Project No.:***	DYAL-19-010
Project Name:	Gilman Springs Rd	***Tested by:***	GA
Project Number:	2018-019	***Checked by:***	KL
Boring Number:	DYB19-13	***Date:***	05/08/19
Sample No:	0 (Bulk)	***Mold size (in):***	6"
Depth (ft) :	0-5	***Procedure:***	C
Soil Description:	Brown, Clayey Sand (SC)	***% Ret. on 3/4":***	0.2



Maximum Dry Density (pcf): 127.8
Optimum Moisture Content (%): 10.4

PLATE
C8

EXPANSION INDEX

ASTM D4829

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

Client:	Diaz Yourman & Associates	***HAI Project No.:***	DYAL-19-010
Project Name:	Gilman Springs Rd	***Apparatus #:***	1
Project No.:	2018-019		
Boring No.:	DYB19-03	***Tested by:***	GA
Sample No.:	Bulk	***Checked by:***	KL
Depth (ft):	0-5	***Date:***	5/8/2019
Soil Description:	Brown, Silty Sand (SM)		

INITIAL SPECIMEN INFO		
Wt. of wet soil + cont.	171.25	g
Wt. of dry soil + cont.	161.98	g
Wt. of container	24.20	g
Wt. of water	9.27	g
Wt. of dry soil	137.78	g
Moisture Content	**6.7**	**%**
Wt. of wet soil + ring	635.66	g
Wt. of ring	206.84	g
Wt. of wet soil	428.82	g
Wet density of soil	130.7	pcf
Dry density of soil	122.5	pcf
Specific gravity of soil	2.68	
Saturation	**49.3**	**%**

FINAL SPECIMEN INFO		
Wt. of wet soil + cont.	658.56	g
Wt. of dry soil + cont.	604.35	g
Wt. of container	206.84	g
Wt. of water	54.21	g
Wt. of dry soil	397.51	g
Moisture Content	**13.6**	**%**

Date & Time	**Elapsed Time (min)**	**Dial Reading**	**Δh, Expansion**
1/0/1900 0:00	0	0	0
1/0/1900 0:10	10	-0.0028	-0.0028
Add Distilled Water to Sample			
1/1/1900 0:00	1440	-0.0028	0.0000

Expansion Index = **0** **Very Low**

PLATE
C9

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

EXPANSION INDEX

ASTM D4829

Client:	Diaz Yourman & Associates	***HAI Project No.:***	DYAL-19-010
Project Name:	Gilman Springs Rd	***Apparatus #:***	2
Project No.:	2018-019		
Boring No.:	DYB19-13	***Tested by:***	GA
Sample No.:	Bulk	***Checked by:***	KL
Depth (ft):	0-5	***Date:***	5/8/2019
Soil Description:	Brown, Clayey Sand (SC)		

INITIAL SPECIMEN INFO		
Wt. of wet soil + cont.	172.55	g
Wt. of dry soil + cont.	161.42	g
Wt. of container	24.19	g
Wt. of water	11.13	g
Wt. of dry soil	137.23	g
Moisture Content	**8.1**	**%**
Wt. of wet soil + ring	612.70	g
Wt. of ring	206.54	g
Wt. of wet soil	406.16	g
Wet density of soil	125.2	pcf
Dry density of soil	115.8	pcf
Specific gravity of soil	2.68	
Saturation	**48.9**	**%**

FINAL SPECIMEN INFO		
Wt. of wet soil + cont.	649.68	g
Wt. of dry soil + cont.	578.34	g
Wt. of container	206.54	g
Wt. of water	71.34	g
Wt. of dry soil	371.80	g
Moisture Content	**19.2**	**%**

Date & Time	Elapsed Time (min)	Dial Reading	Δh, Expansion
1/0/1900 0:00	0	0	0
1/0/1900 0:10	10	-0.0005	-0.0005
Add Distilled Water to Sample			
1/1/1900 0:00	1440	0.0424	0.0429

Expansion Index = 43 Low

PLATE
C10

SAND EQUIVALENT TEST
CTM 217

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

Client: Diaz Yourman & Associates
Project Name: Gilman Springs Rd
Project No.: 2018-019
Boring No.: DYB19-01
Sample No.: 0 (Bulk)
Soil Description: Dark Brown, Clayey Sand (SC)

HAI Project No.: DYAL-19-010
Tested by: GA
Checked by: KL/MJ
Date: 05/08/19

T1	T2	T3	T4	R1	R2	SE	Average SE
9:50	10:00	10:01	10:21	14.00	2.20	16	**16**
9:53	10:03	10:05	10:25	13.90	2.20	16	

T1 = Starting Time
T2 = (T1 + 10 min) Begin Agitation
(100 cycles in 30 sec)

T3 = Settlement Starting Time
T4 = (T3 + 20 min) Take Clay Reading (R1)
and Sand Reading (R2)

Sand Equivalent = R2 / R1 * 100
Record SE as Next Higher Integer

SAND EQUIVALENT TEST
CTM 217

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

Client: Diaz Yourman & Associates
Project Name: Gilman Springs Rd
Project No.: 2018-019
Boring No.: DYB19-07
Sample No.: 0 (Bulk)
Soil Description: Brown, Silty Sand with Gravel (SM)

HAI Project No.: DYAL-19-010
Tested by: GA
Checked by: KL/MJ
Date: 05/08/19

T1	T2	T3	T4	R1	R2	SE	Average SE
9:56	10:06	10:07	10:27	12.3	2.5	21	**21**
9:59	10:09	10:10	10:30	12.1	2.5	21	

T1 = Starting Time
T2 = (T1 + 10 min) Begin Agitation
(100 cycles in 30 sec)

T3 = Settlement Starting Time
T4 = (T3 + 20 min) Take Clay Reading (R1)
and Sand Reading (R2)

Sand Equivalent = R2 / R1 * 100
Record SE as Next Higher Integer

PLATE
C12

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

SAND EQUIVALENT TEST
CTM 217

Client: Diaz Yourman & Associates
Project Name: Gilman Springs Rd
Project No.: 2018-019
Boring No.: DYB19-13
Sample No.: S0 (Bulk)
Soil Description: Brown, Silty Sand (SM)

HAI Project No.: DYAL-19-010
Tested by: GA
Checked by: KL/MJ
Date: 05/08/19

T1	T2	T3	T4	R1	R2	SE	Average SE
10:03	10:13	10:14	10:34	14.0	0.9	7	7
10:05	10:15	10:17	10:37	14.1	0.9	7	

T1 = Starting Time
T2 = (T1 + 10 min) Begin Agitation
(100 cycles in 30 sec)

T3 = Settlement Starting Time
T4 = (T3 + 20 min) Take Clay Reading (R1)
and Sand Reading (R2)

Sand Equivalent = R2 / R1 * 100
Record SE as Next Higher Integer

PLATE
C13

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

SAND EQUIVALENT TEST
CTM 217

Client: Diaz Yourman & Associates
Project Name: Gilman Springs Rd
Project No.: 2018-019
Boring No.: DYB19-08
Sample No.: 0 (Bulk)
Soil Description: Brown, Silty Sand (SM)

HAI Project No.: DYAL-19-010
Tested by: GA
Checked by: KL/MJ
Date: 05/08/19

T1	T2	T3	T4	R1	R2	SE	Average SE
2:38	2:48	2:49	3:09	10.7	3.7	35	
2:41	2:51	2:52	3:12	9.9	3.4	35	**35**
2:44	2:54	2:55	3:15	10.7	3.7	35	

T1 = Starting Time
T2 = (T1 + 10 min) Begin Agitation
(100 cycles in 30 sec)

T3 = Settlement Starting Time
T4 = (T3 + 20 min) Take Clay Reading (R1)
and Sand Reading (R2)

Sand Equivalent = R2 / R1 * 100
Record SE as Next Higher Integer

DIRECT SHEAR TEST

ASTM D3080

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

HAI Pr No.: DYAL-19-010

Client: Diaz Yourman & Associates
Project Name: Gilman Springs Rd
Project Number: 2018-019
Boring No.: DYB19-09
Sample No.: 4
Sample Type: Undisturbed Ring
Depth (ft): 15
Soil description: Brown, Silty Sand (SM)

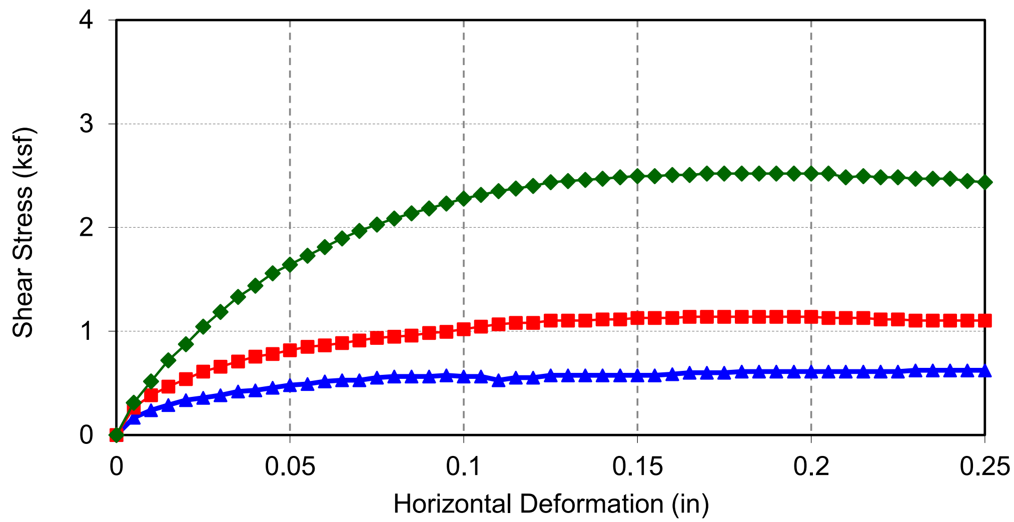
Tested by: AH
Checked by: KL
Date: 5/8/2019

Type of test: **Consolidated, Drained**

Test No.	1	2	3
Symbol	▲	■	◆
Normal Stress (ksf)	**1**	**2**	**4**
Deformation Rate (in/min)	**0.002**	**0.002**	**0.002**

Peak Shear Stress (ksf)	**O**	**0.62**	**1.14**	**2.52**
Shear Stress @ End of Test (ksf)	**X**	**0.62**	**1.10**	**2.44**

Initial Height of Sample (in)	1.000	1.000	1.000
Height of Sample before Shear (in)	0.9840	0.9789	0.9295
Diameter of Sample (in)	2.416	2.416	2.416
Initial Moisture Content (%)	**4.9**	**4.9**	**4.9**
Final Moisture Content (%)	**17.7**	**17.8**	**16.5**
Dry Density (pcf)	**104.6**	**105.2**	**102.0**



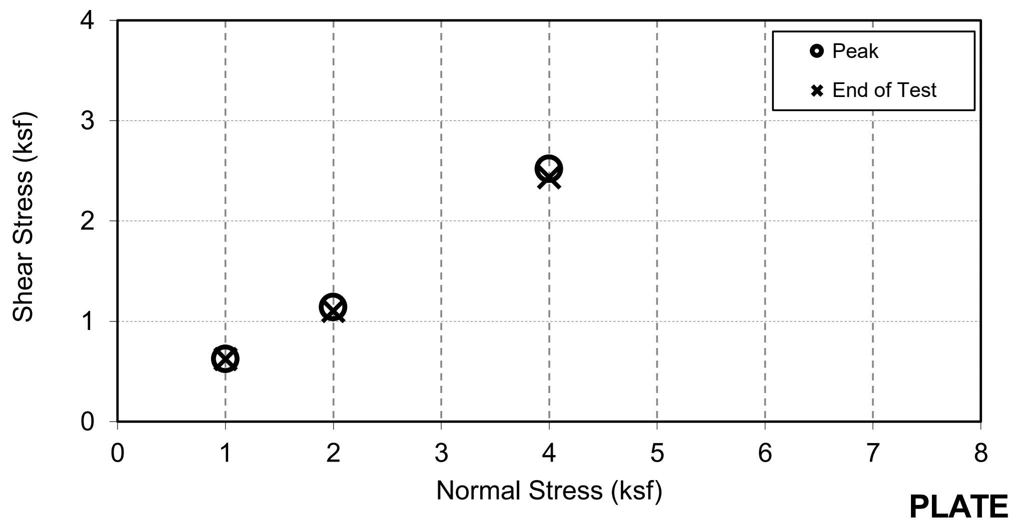


PLATE
C15

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

DIRECT SHEAR TEST

ASTM D3080

HAI Pr No.: DYAL-19-010

Client: Diaz Yourman & Associates
Project Name: Gilman Springs Rd
Project Number: 2018-019
Boring No.: DYB19-10
Sample No.: 3
Sample Type: Undisturbed Ring
Depth (ft): 10
Soil description: Brown, Poorly Graded Sand with Gravel (SP)

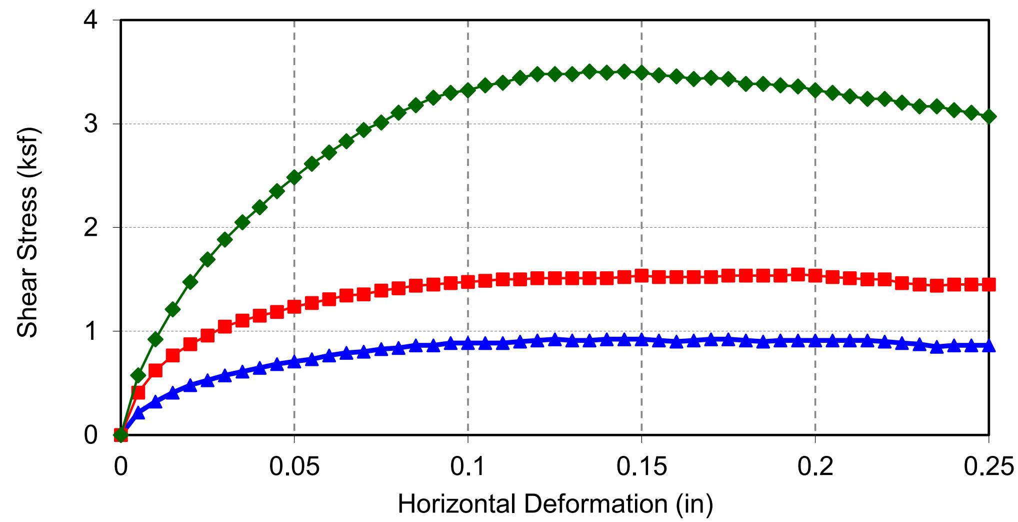
Tested by: AH
Checked by: KL
Date: 5/8/2019

Type of test: **Consolidated, Drained**

Test No.	1	2	3
Symbol	▲	■	◆
Normal Stress (ksf)	**1**	**2**	**4**
Deformation Rate (in/min)	**0.002**	**0.002**	**0.002**

Peak Shear Stress (ksf)	**O**	**0.92**	**1.55**	**3.50**
Shear Stress @ End of Test (ksf)	**X**	**0.86**	**1.45**	**3.07**

Initial Height of Sample (in)	1.000	1.000	1.000
Height of Sample before Shear (in)	0.9879	0.9840	0.9749
Diameter of Sample (in)	2.416	2.416	2.416
Initial Moisture Content (%)	**0.8**	**0.8**	**0.8**
Final Moisture Content (%)	**15.3**	**14.5**	**14.2**
Dry Density (pcf)	**112.1**	**112.7**	**116.5**



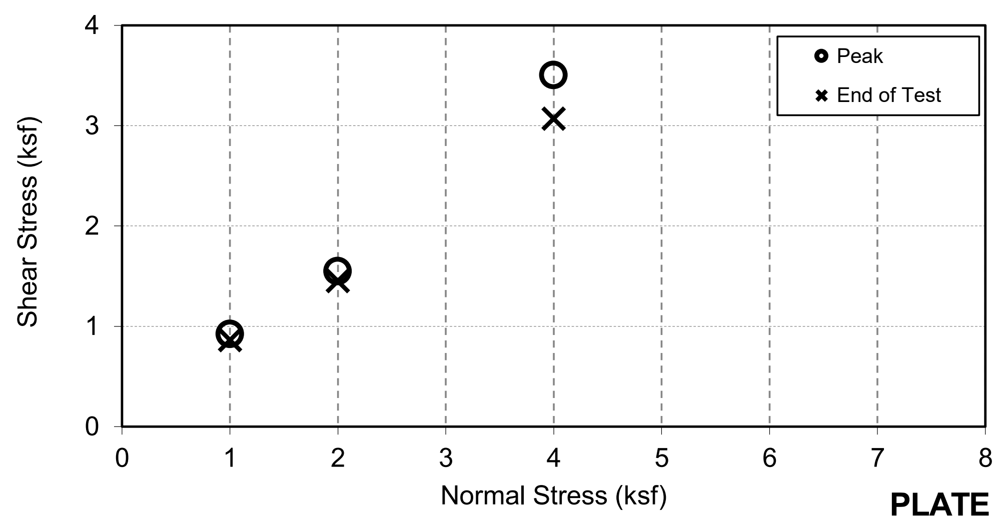


PLATE
C16

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

DIRECT SHEAR TEST

ASTM D3080

HAI Pr No.: DYAL-19-010

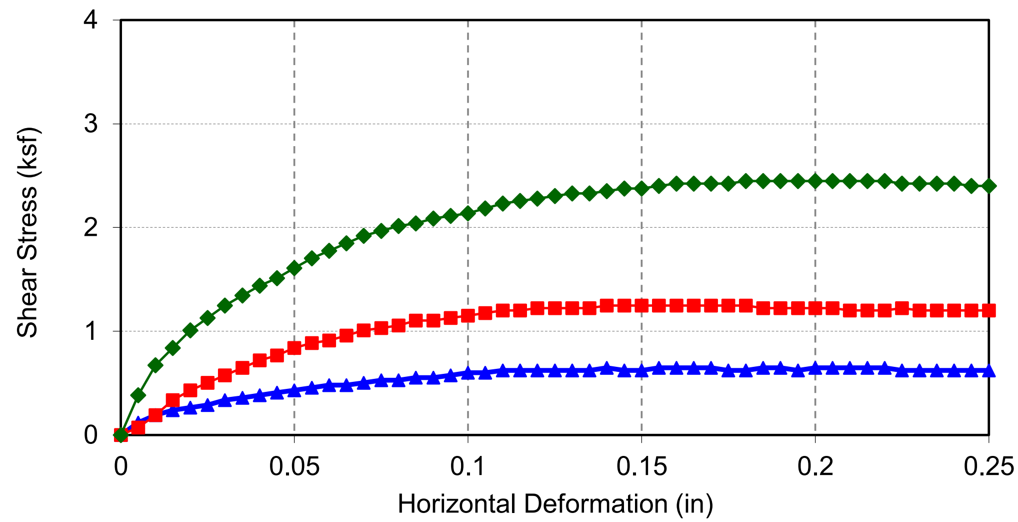
Client:	Diaz Yourman & Associates
Project Name:	Gilman Springs Rd
Project Number:	2018-019
Boring No.:	DYB19-10
Sample No.:	5
Sample Type:	Undisturbed Ring
Depth (ft):	20
Soil description:	Light Brown, Silty Sand (SM)
Type of test:	**Consolidated, Drained**

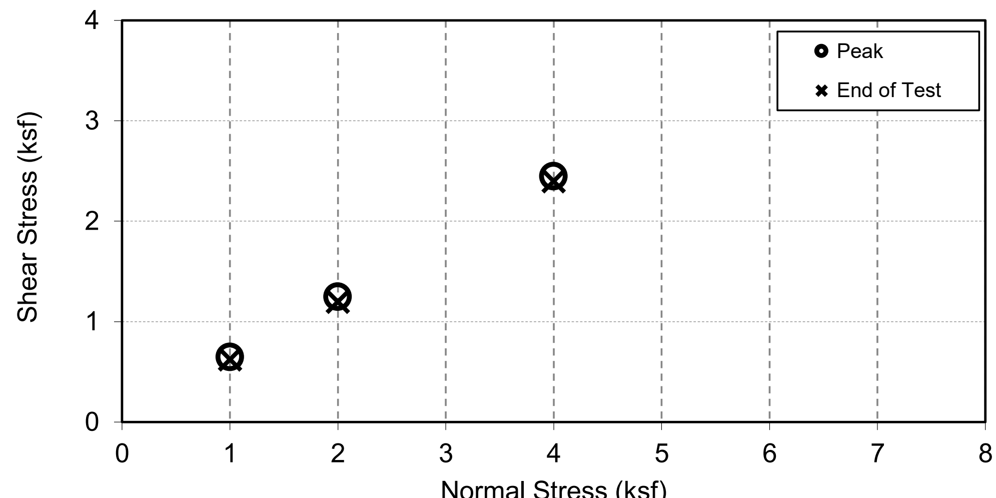
Tested by: AH
Checked by: KL
Date: 5/8/2019

Test No.	1	2	3
Symbol	▲ (blue triangle)	■ (red square)	◆ (green diamond)
Normal Stress (ksf)	**1**	**2**	**4**
Deformation Rate (in/min)	**0.002**	**0.002**	**0.002**

Peak Shear Stress (ksf)	**O**	**0.65**	**1.25**	**2.45**
Shear Stress @ End of Test (ksf)	**X**	**0.62**	**1.20**	**2.40**

Initial Height of Sample (in)	1.000	1.000	1.000
Height of Sample before Shear (in)	0.9602	0.9498	0.9286
Diameter of Sample (in)	2.416	2.416	2.416
Initial Moisture Content (%)	**2.8**	**2.8**	**2.8**
Final Moisture Content (%)	**19.0**	**17.6**	**17.1**
Dry Density (pcf)	**103.6**	**105.0**	**104.0**





PLATE

C17

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

DIRECT SHEAR TEST
ASTM D3080

HAI Pr No.: DYAL-19-010

Client: Diaz Yourman & Associates
Project Name: Gilman Springs Rd
Project Number: 2018-019
Boring No.: DYB19-11
Sample No.: 3
Sample Type: Undisturbed Ring
Depth (ft): 10
Soil description: Brown, Silty Sand with Gravel (SM)

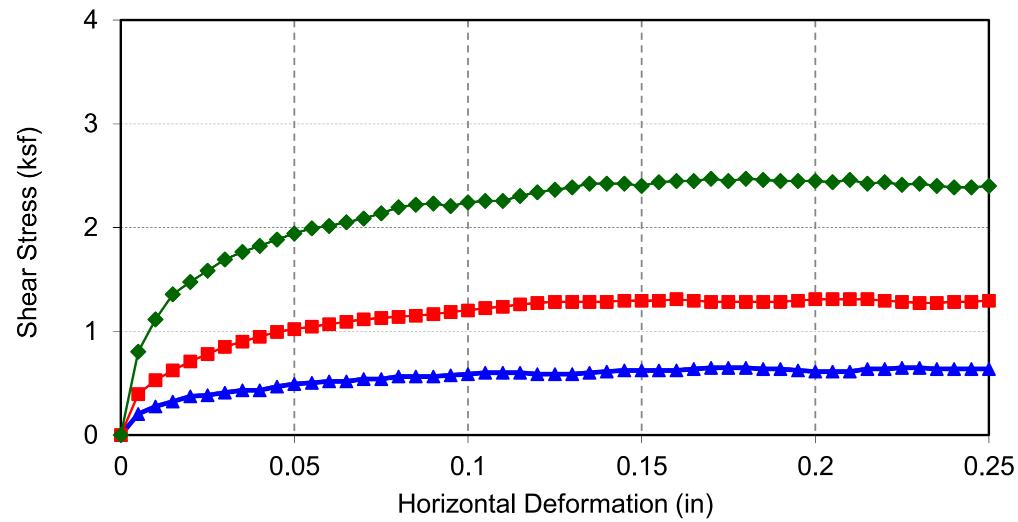
Tested by: AH
Checked by: KL
Date: 5/8/2019

Type of test: **Consolidated, Drained**

Test No.	1	2	3
Symbol	▲ (blue)	■ (red)	◆ (green)
Normal Stress (ksf)	**1**	**2**	**4**
Deformation Rate (in/min)	**0.002**	**0.002**	**0.002**

Peak Shear Stress (ksf)	**O**	**0.65**	**1.31**	**2.47**
Shear Stress @ End of Test (ksf)	**X**	**0.64**	**1.30**	**2.40**

Initial Height of Sample (in)	1.000	1.000	1.000
Height of Sample before Shear (in)	0.9854	0.9837	0.9758
Diameter of Sample (in)	2.416	2.416	2.416
Initial Moisture Content (%)	**3.0**	**3.0**	**3.0**
Final Moisture Content (%)	**17.5**	**15.1**	**16.0**
Dry Density (pcf)	**94.7**	**101.9**	**103.9**



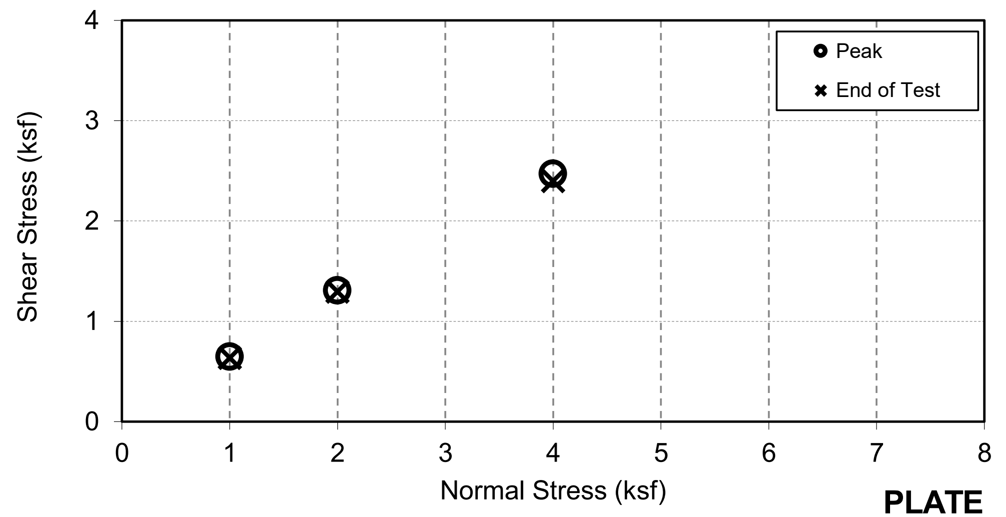


PLATE
C18

DIRECT SHEAR TEST

ASTM D3080

HAI Pr No.: DYAL-19-010

Client: Diaz Yourman & Associates
Project Name: Gilman Springs Rd
Project Number: 2018-019
Boring No.: DYB19-11
Sample No.: 5
Sample Type: Undisturbed Ring
Depth (ft): 23
Soil description: Light Brown, Silty Sand with Gravel (SM)

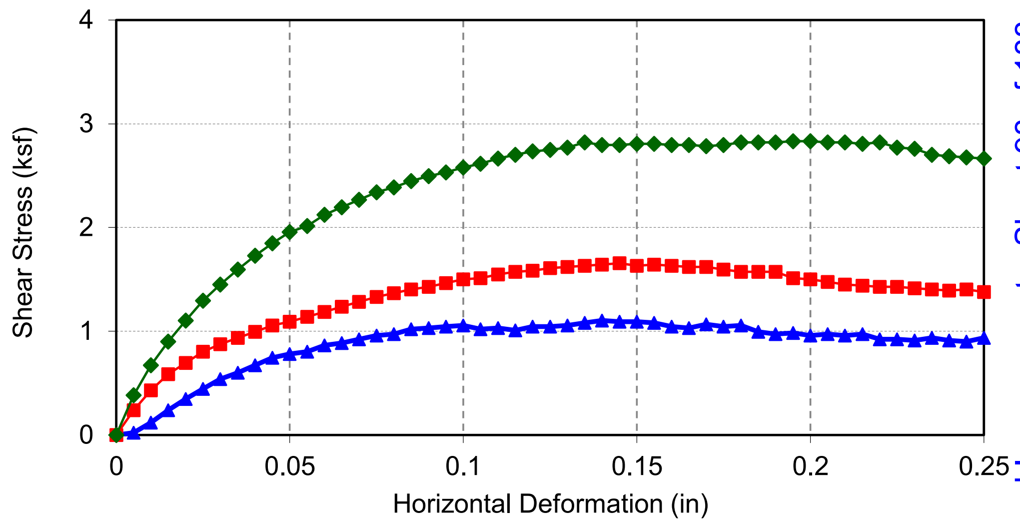
Tested by: AH
Checked by: KL
Date: 5/8/2019

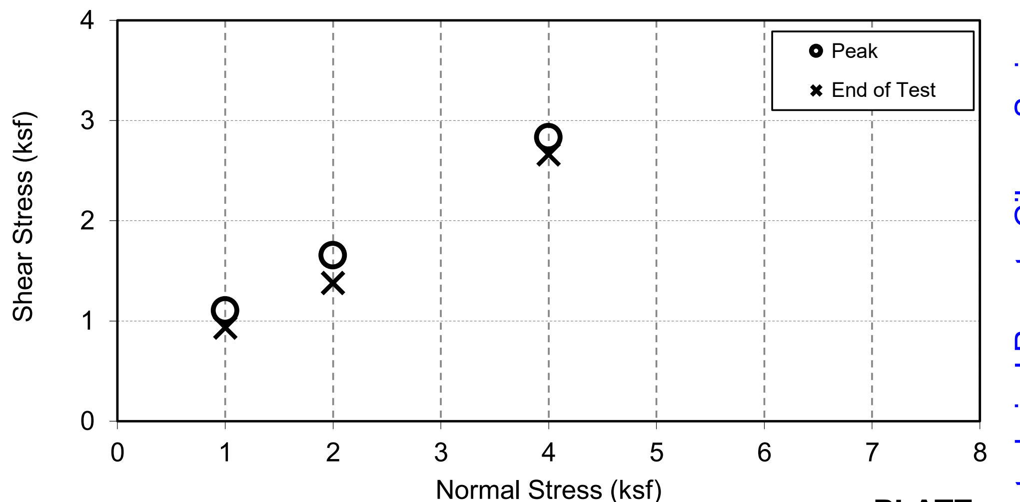
Type of test: **Consolidated, Drained**

Test No.	1	2	3
Symbol	▲ (blue)	■ (red)	◆ (green)
Normal Stress (ksf)	**1**	**2**	**4**
Deformation Rate (in/min)	**0.002**	**0.002**	**0.002**

Peak Shear Stress (ksf)	**O**	**1.10**	**1.66**	**2.83**
Shear Stress @ End of Test (ksf)	**X**	**0.94**	**1.38**	**2.66**

Initial Height of Sample (in)	1.000	1.000	1.000
Height of Sample before Shear (in)	0.9940	0.9937	0.9922
Diameter of Sample (in)	2.416	2.416	2.416
Initial Moisture Content (%)	**0.9**	**0.9**	**0.9**
Final Moisture Content (%)	**13.7**	**13.3**	**14.3**
Dry Density (pcf)	**113.2**	**113.6**	**111.9**





PLATE

C19

• ANALYSIS
• DESIGN

LaBelle • Marvin

PROFESSIONAL PAVEMENT ENGINEERING

A CALIFORNIA CORPORATION

• SOILS, ASPHALT TECHNOLOGY

May 17, 2019

Kang Chieh Lin
Hushmand Associates, Inc.
250 Goddard
Irvine, California 92618

Project No. 44901

Attention: Kang Chieh Lin

Testing of the bulk soil samples delivered to our laboratory on 5/13/2019 has been completed.

Project No.:	**DYAL 18-010 / 2019-019**
Reference:	**Gilman Springs Road**
Samples:	**DYB19-02 @ 0'-5'**
	DYB19-03 @ 0'-5'
	DYB19-07 @ 0'-5'
	DYB19-13 @ 0'-5'

Data sheets are attached for your use and file. Any untested portion of the sample will be retained for a period of 60 days prior to disposal. The opportunity to be of service is sincerely appreciated and should you have any questions, kindly call.

Very truly yours,

REGISTERED PROFESSIONAL ENGINEER
STEVEN R. MARVIN
NO. 30659
CIVIL
STATE OF CALIFORNIA

Steven R. Marvin
RCE 30659

SRM:tw
Enclosure

PLATE
C20

2700 S. GRAND AVENUE • SANTA ANA, CA 92705-5404 • (714) 546-3468 • FAX (714) 546-5841
INFO@LABELLEMARVIN.COM

LaBelle Marvin

R - VALUE DATA SHEET

PROJECT No. 44901

DATE: 5/17/2019

BORING NO. DYB19-02 @ 0'-5'

Gilman Springs Road

P.N. DYAL 18-010/2018-019

SAMPLE DESCRIPTION: Brown Gravelly Sandy Clay

R-VALUE TESTING DATA | CA TEST 301

	SPECIMEN ID		
	a	b	c
Mold ID Number	1	2	3
Water added, grams	78	48	29
Initial Test Water, %	14.4	11.5	9.7
Compact Gage Pressure,psi	40	70	160
Exudation Pressure, psi	150	357	537
Height Sample, Inches	2.66	2.54	2.45
Gross Weight Mold, grams	3141	3120	3092
Tare Weight Mold, grams	1946	1956	1949
Sample Wet Weight, grams	1195	1164	1143
Expansion, Inches x 10exp-4	0	36	90
Stability 2,000 lbs (160psi)	58 / 135	31 / 78	22 / 51
Turns Displacement	4.23	4.18	3.87
R-Value Uncorrected	10	39	58
R-Value Corrected	11	39	58
Dry Density, pcf	119.0	124.5	128.8

DESIGN CALCULATION DATA

Traffic Index Assumed:	4.0	4.0	4.0
G.E. by Stability	0.91	0.62	0.43
G. E. by Expansion	0.00	1.20	3.00

Equilibrium R-Value	**31** by EXPANSION	Examined & Checked: 5 /17/ 19
REMARKS:	Gf = 1.25 3.8% Retained on the 3/4" Sieve.	Steven R. Marvin, RCE 30659

REGISTERED PROFESSIONAL ENGINEER STEVEN R. MARVIN C 30659 CIVIL STATE OF CALIFORNIA

PLATE C21

The data above is based upon processing and testing samples as received from the field. Test procedures in accordance with latest revisions to Department of Transportation, State of California, Materials & Research Test Method No. 301.

R-VALUE GRAPHICAL PRESENTATION

LaBelle Marvin

PROJECT NO. 44901

DATE: 5 /17/ 19

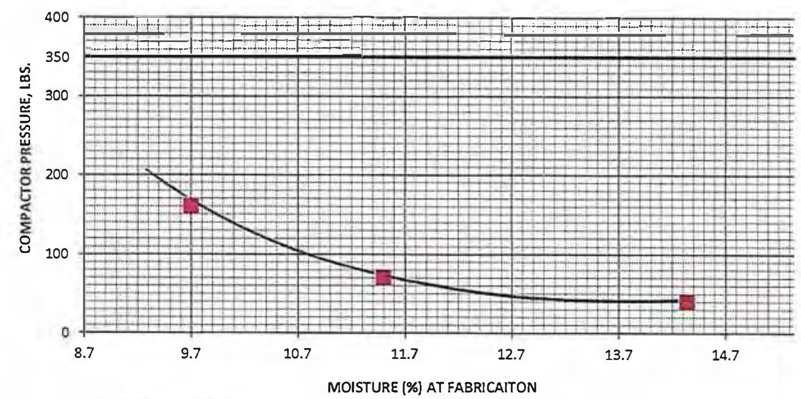
REMARKS:

BORING NO. DYB19-02 @ 0'-5'

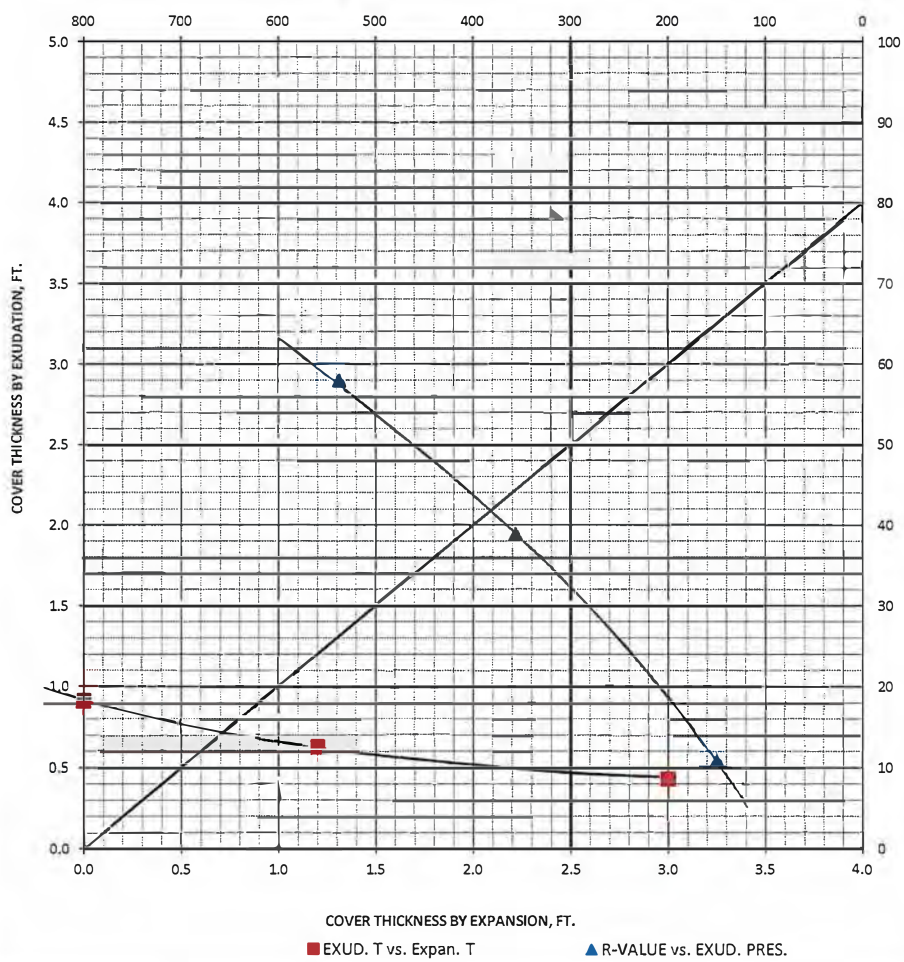
Gilman Springs Road

P.N. DYAL 18-010/2018-019

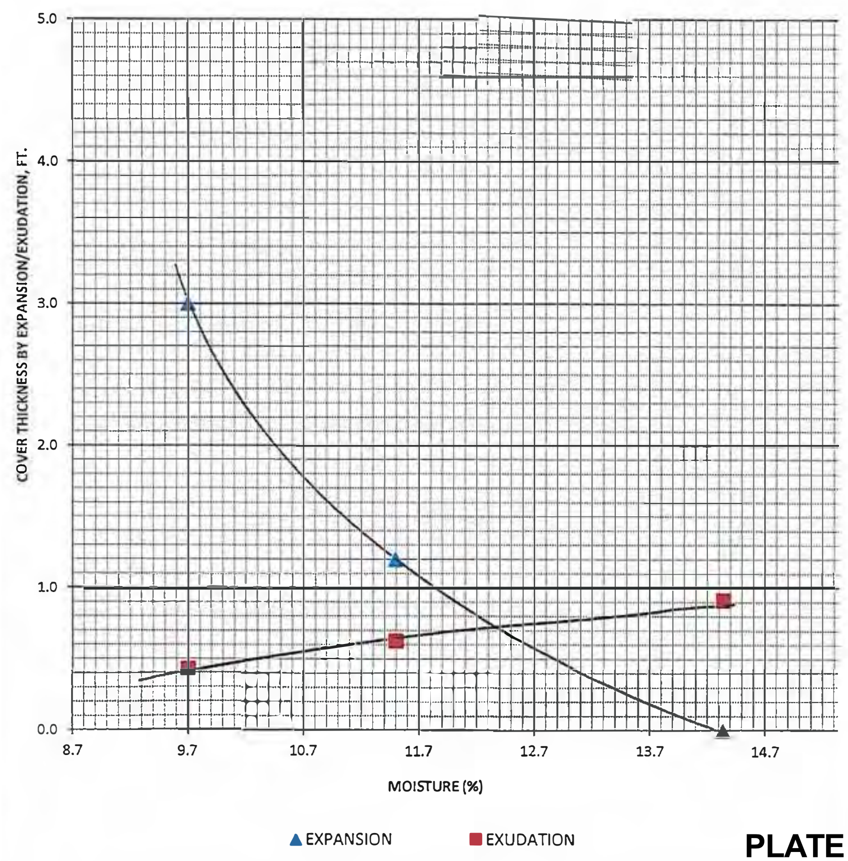
COMPACTOR PRESSURE vs MOISTURE %



COVER THICKNESS BY EXUDATION vs COVER THICKNESS BY EXPANSION



COVER THICKNESS vs MOISTURE %



PLATE

C22

R - VALUE DATA SHEET

LaBelle Marvin

PROJECT No. 44901
DATE: 5/17/2019

BORING NO. DYB19-03 @ 0'-5'
Gilman Springs Road
P.N. DYAL 18-010/2018-019

SAMPLE DESCRIPTION: BrownSandy Silt

R-VALUE TESTING DATA | CA TEST 301

	SPECIMEN ID		
	a	b	c
Mold ID Number	4	5	6
Water added, grams	35	50	28
Initial Test Water, %	8.3	9.7	7.6
Compact Gage Pressure,psi	250	150	350
Exudation Pressure, psi	429	188	707
Height Sample, Inches	2.50	2.55	2.48
Gross Weight Mold, grams	3119	3124	3114
Tare Weight Mold, grams	1959	1960	1958
Sample Wet Weight, grams	1160	1164	1156
Expansion, Inches x 10exp-4	18	4	21
Stability 2,000 lbs (160psi)	14 / 28	19 / 38	12 / 22
Turns Displacement	4.56	5.22	3.96
R-Value Uncorrected	72	61	80
R-Value Corrected	72	61	80
Dry Density, pcf	129.8	126.1	131.2

DESIGN CALCULATION DATA

Traffic Index Assumed:	4.0	4.0	4.0
G.E. by Stability	0.29	0.40	0.20
G. E. by Expansion	0.60	0.13	0.70

Equilibrium R-Value	**62** by EXPANSION	Examined & Checked: 5 /17/ 19
REMARKS:	Gf = 1.25 2.8% Retained on the 3/4" Sieve.	Steven R. Marvin, RCE 30659

REGISTERED PROFESSIONAL ENGINEER STEVEN R. MARVIN C 30659 CIVIL STATE OF CALIFORNIA

The data above is based upon processing and testing samples as received from the field. Test procedures in accordance with latest revisions to Department of Transportation, State of California, Materials & Research Test Method No. 301.

PLATE
C23

LaBelle Marvin

R-VALUE GRAPHICAL PRESENTATION

PROJECT NO. 44901

DATE: 5 /17/ 19

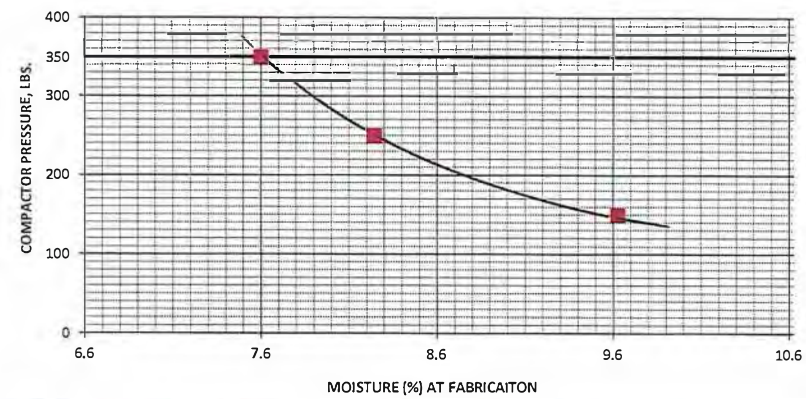
REMARKS:

BORING NO. DYB19-03 @ 0'-5'

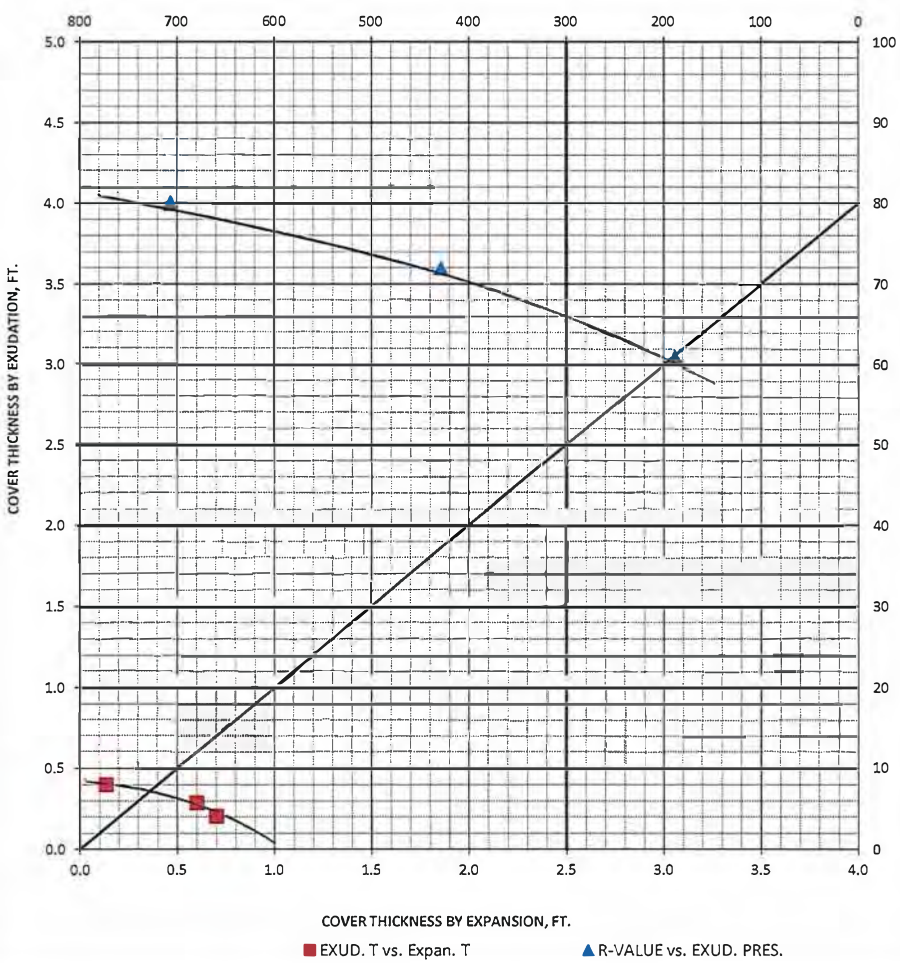
Gilman Springs Road

P.N. DYAL 18-010/2018-019

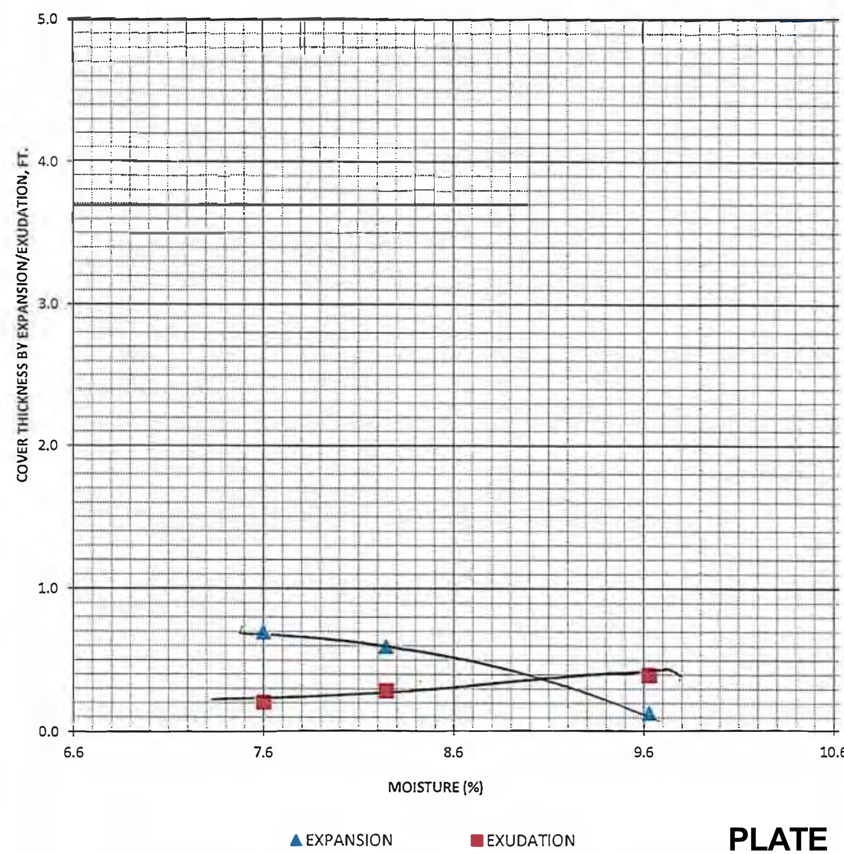
COMPACTOR PRESSURE vs MOISTURE %



COVER THICKNESS BY EXUDATION vs COVER THICKNESS BY EXPANSION



COVER THICKNESS vs MOISTURE %



PLATE

C24

R - VALUE DATA SHEET

LaBelle Marvin

PROJECT No. 44901

DATE: 5/16/2019

BORING NO. DYB19-07 @ 0'-5'
Gilman Springs Road
P.N. DYAL 18-010/2018-019

SAMPLE DESCRIPTION: Brown Gravelly Sandy Silt

R-VALUE TESTING DATA | CA TEST 301			
	SPECIMEN ID		
	a	b	c
Mold ID Number	10	11	12
Water added, grams	40	53	46
Initial Test Water, %	7.5	8.7	8.1
Compact Gage Pressure,psi	310	180	250
Exudation Pressure, psi	601	194	364
Height Sample, Inches	2.49	2.53	2.51
Gross Weight Mold, grams	3100	3122	3109
Tare Weight Mold, grams	1947	1952	1948
Sample Wet Weight, grams	1153	1170	1161
Expansion, Inches x 10exp-4	20	5	9
Stability 2,000 lbs (160psi)	13 / 23	17 / 32	14 / 27
Turns Displacement	4.15	4.87	4.33
R-Value Uncorrected	78	67	74
R-Value Corrected	78	67	74
Dry Density, pcf	130.5	128.9	129.7

DESIGN CALCULATION DATA

Traffic Index Assumed:	4.0	4.0	4.0
G.E. by Stability	0.23	0.34	0.27
G. E. by Expansion	0.67	0.17	0.30

Equilibrium R-Value	**71** by EXUDATION	Examined & Checked: 5 /16/ 19
REMARKS:	Gf = 1.25 3.2% Retained on the 3/4" Sieve.	

REGISTERED PROFESSIONAL ENGINEER STEVEN R. MARVIN C 30659 CIVIL STATE OF CALIFORNIA

Steven R. Marvin, RCE 30659

The data above is based upon processing and testing samples as received from the field. Test procedures in accordance with latest revisions to Department of Transportation, State of California, Materials & Research Test Method No. 301.

PLATE
C25

R-VALUE GRAPHICAL PRESENTATION

LaBelle Marvin

PROJECT NO. 44901

DATE: 5 /16/ 19

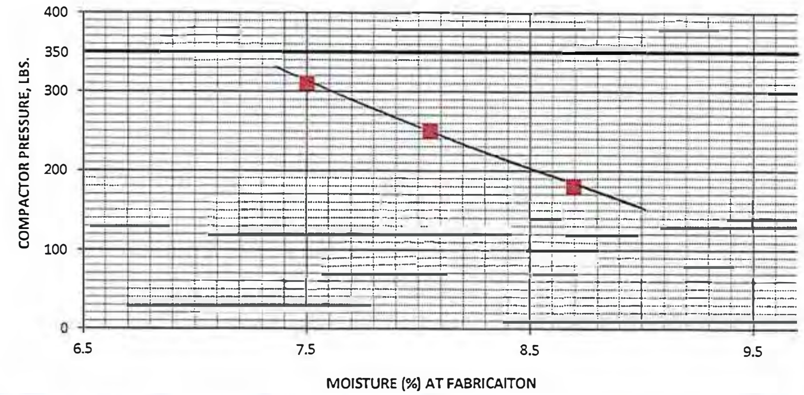
REMARKS:

BORING NO. DYB19-07 @ 0'-5'

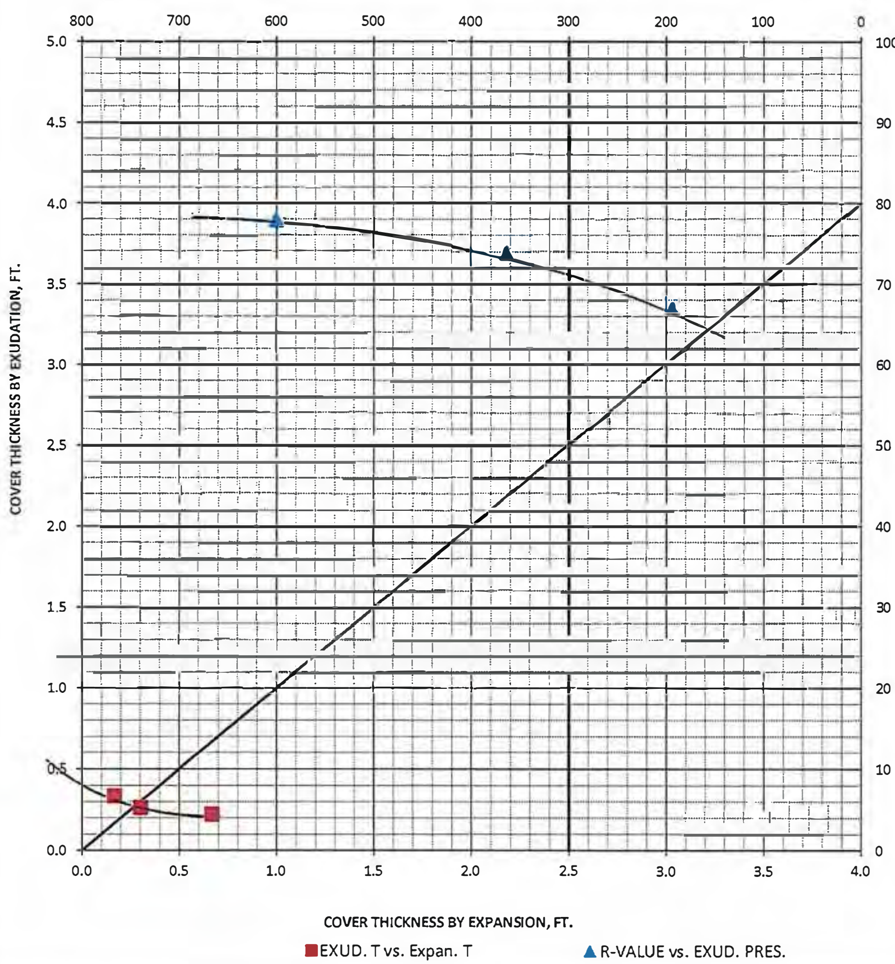
Gilman Springs Road

P.N. DYAL 18-010/2018-019

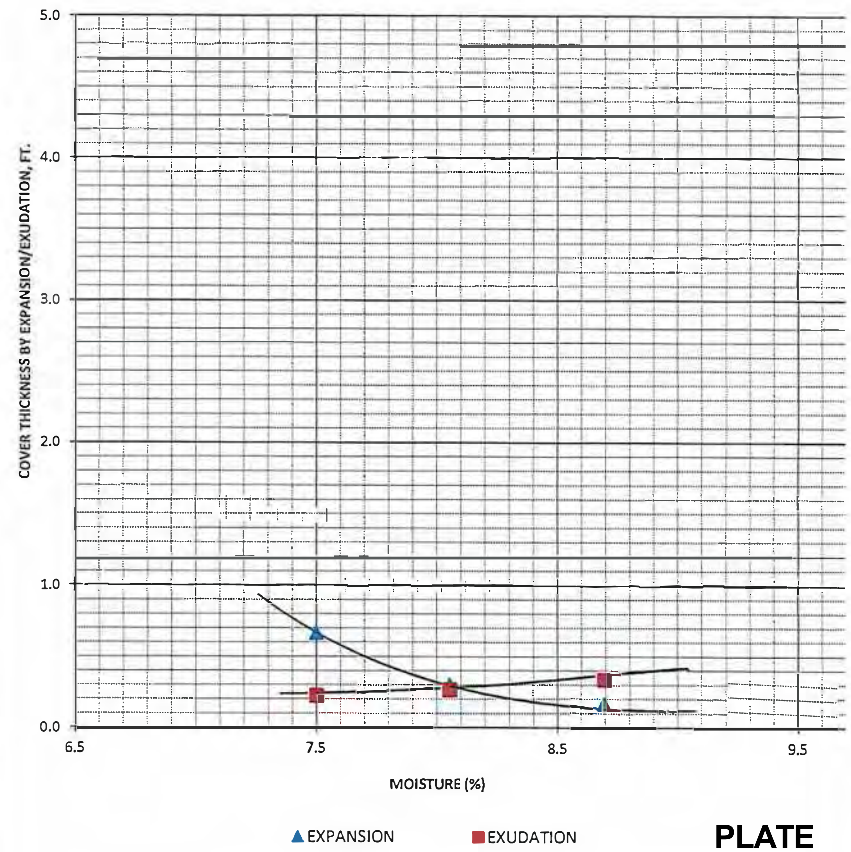
COMPACTOR PRESSURE vs MOISTURE %



COVER THICKNESS BY EXUDATION vs COVER THICKNESS BY EXPANSION



COVER THICKNESS vs MOISTURE %



PLATE

C26

LaBelle Marvin

R - VALUE DATA SHEET

PROJECT No. 44901

DATE: 5/16/2019

BORING NO. DYB19-13 @ 0'-5'

Gilman Springs Road

P.N. DYAL 18-010/2018-019

SAMPLE DESCRIPTION: Brown Sandy Clay

R-VALUE TESTING DATA | CA TEST 301			
	SPECIMEN ID		
	a	b	c
Mold ID Number	7	8	9
Water added, grams	70	45	27
Initial Test Water, %	16.0	13.5	11.7
Compact Gage Pressure,psi	40	110	180
Exudation Pressure, psi	175	283	495
Height Sample, Inches	2.63	2.51	2.42
Gross Weight Mold, grams	3094	3060	2869
Tare Weight Mold, grams	1955	1950	1775
Sample Wet Weight, grams	1139	1110	1094
Expansion, Inches x 10exp-4	16	49	79
Stability 2,000 lbs (160psi)	52 / 125	36 / 85	29 / 62
Turns Displacement	4.62	4.49	3.79
R-Value Uncorrected	13	33	51
R-Value Corrected	14	33	49
Dry Density, pcf	113.1	118.0	122.6

DESIGN CALCULATION DATA

Traffic Index Assumed:	4.0	4.0	4.0
G.E. by Stability	0.88	0.69	0.52
G. E. by Expansion	0.53	1.63	2.63

Equilibrium R-Value	**21** by EXPANSION	Examined & Checked: 5 /16/ 19

REMARKS:

Gf = 1.25

0.3% Retained on the

3/4" Sieve.

REGISTERED PROFESSIONAL ENGINEER STEVEN R. MARVIN C 30659 STATE OF CALIFORNIA

Steven R. Marvin, RCE 30659

The data above is based upon processing and testing samples as received from the field. Test procedures in accordance with latest revisions to Department of Transportation, State of California, Materials & Research Test Method No. 301.

PLATE

C27

R-VALUE GRAPHICAL PRESENTATION

LaBelle Marvin

PROJECT NO. 44901

DATE: 5 /16/ 19

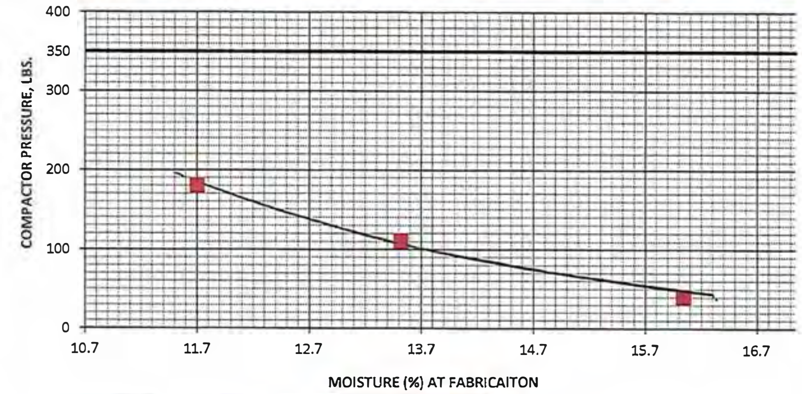
REMARKS:

BORING NO. DYB19-13 @ 0'-5'

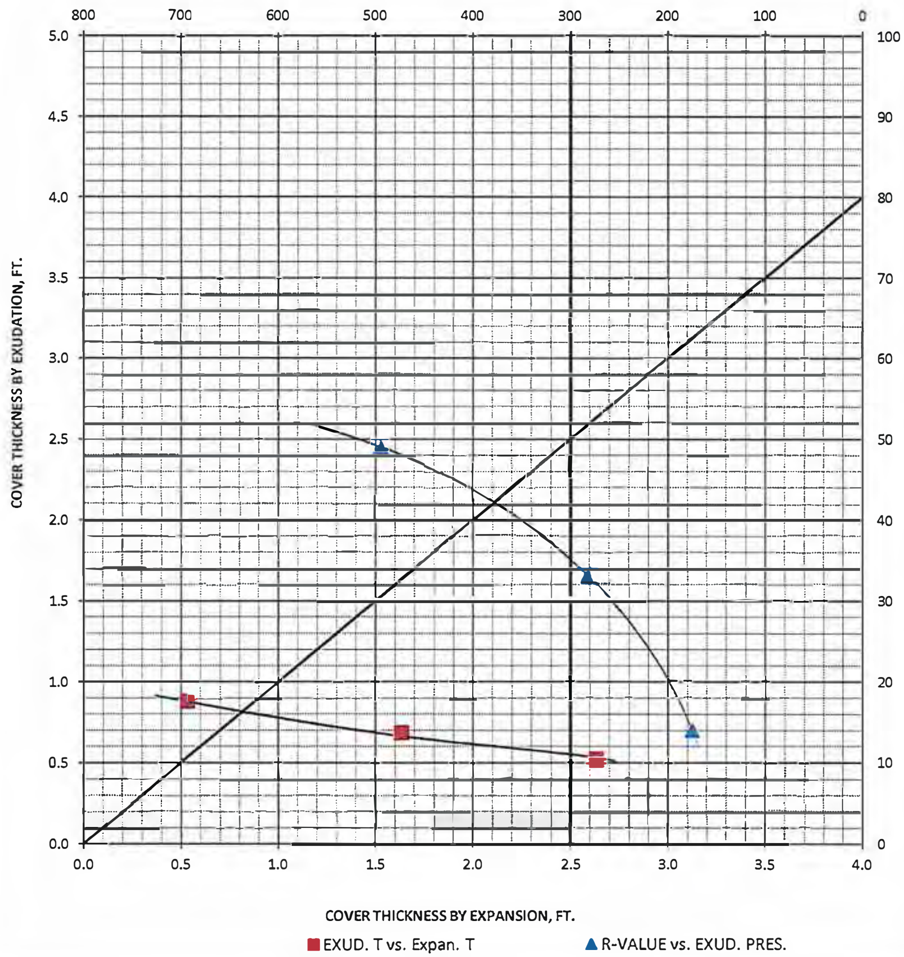
Gilman Springs Road

P.N. DYAL 18-010/2018-019

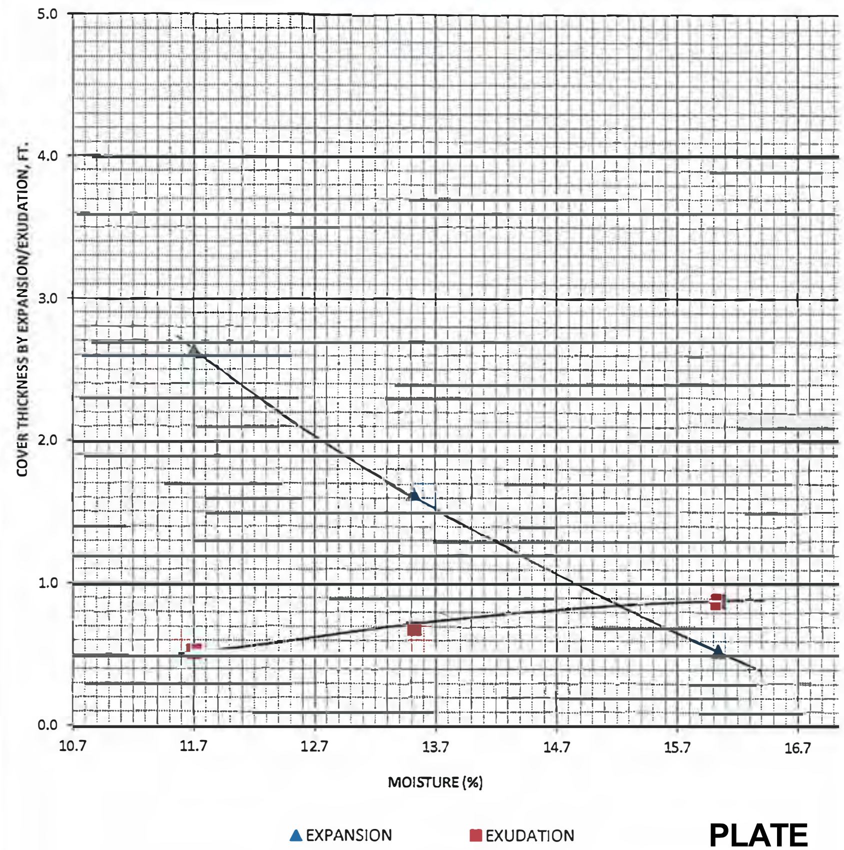
COMPACTOR PRESSURE vs MOISTURE %



COVER THICKNESS BY EXUDATION vs COVER THICKNESS BY EXPANSION



COVER THICKNESS vs MOISTURE %



PLATE

C28

Soil Analysis Lab Results

Client: HAI
Job Name: Gilman Springs Rd
Client Job Number: DYAL-19-010 / 2018-019
Project X Job Number: S190514B
May 17, 2019

	Method	ASTM G187		ASTM D4327		ASTM D4327		ASTM G51
Bore# / Description	**Depth**	**Resistivity**		**Sulfates**		**Chlorides**		**pH**
		As Rec'd	Minimum					
	(ft)	(Ohm-cm)	(Ohm-cm)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	
DYB19-03 0 Bulk	0.0-5.0	20,100	1,608	85.9	0.0086	57.1	0.0057	8.12
DYB19-06 S1~S4 Mixed	1.0-5.0	7,370	1,340	187	0.0187	42.3	0.0042	8.51
DYB19-09 1~4 Mixed	2.5-15.0	6,700	4,221	48.4	0.0048	12.3	0.0012	8.55
DYB19-10 1~4 Mixed	2.5-13.0	22,110	9,380	9.2	0.0009	1.2	0.0001	9.06
DYB19-11 1~4 Mixed	2.5-17.0	20,100	7,370	15.8	0.0016	3.6	0.0004	9.07
DYB19-13 S0 Bulk	0.0-5.0	1,139	670	270	0.0270	471.8	0.0472	9.17
CS-01 Bulk	0.0-2.0	18,760	5,896	16	0.0016	1	0.0001	8.16
CS-02 Bulk	0.0-2.0	8,040	4,891	3.3	0.0003	1	0.0001	8.39
CS-03 Bulk	0.0-2.0	9,380	1,675	32.5	0.0033	3	0.0003	7.82
CS-04 Bulk	0.0-2.0	7,370	2,345	27.7	0.0028	1	0.0001	8.56
CS-05 Bulk	0.0-2.0	3,752	1,943	6.9	0.0007	3	0.0003	8.06
CS-06 Bulk	0.0-2.0	3,149	1,876	5.6	0.0006	2	0.0002	7.84
CS-07 Bulk	0.0-2.0	6,700	2,211	7.2	0.0007	3	0.0003	7.29
CS-08 Bulk	0.0-2.0	14,070	2,881	11.5	0.0012	2	0.0002	8.03

Unk = Unknown
NT = Not Tested
ND = 0 = Not Detected
mg/kg = milligrams per kilogram (parts per million) of dry soil weight
Chemical Analysis performed on 1:3 Soil-To-Water extract
Anions and Cations tested via Ion Chromatograph except Sulfide.

Please call if you have any questions.

Nathan Jacob
Lab Technician

Respectfully Submitted,

Eddie Hernandez, M.Sc., P.E.
Sr. Corrosion Consultant
NACE Corrosion Technologist #16592
Professional Engineer
California No. M37102
ehernandez@projectxcorrosion.com

REGISTERED PROFESSIONAL ENGINEER EDUARDO HERNANDEZ No. M37102 MECHANICAL STATE OF CALIFORNIA

PLATE
C29

APPENDIX D -
GEOLOGICAL EVALUATION MEMO

TECHNICAL MEMORANDUM

GILMAN SPRINGS ROAD IMPROVEMENT PROJECT--ENGINEERING GEOLOGY SLOPE EVALUATION ALONG THE NORTH EDGE OF GILMAN SPRINGS ROAD, RIVERSIDE COUNTY, CALIFORNIA

Prepared for:

Diaz-Yourman & Associates
1616 E. Seventeenth Street
Santa Ana, CA 92705-8509

Prepared by:

Wilson Geosciences Inc.
Altadena, California 91001-2117
626 791-1589
wilsongeosciencesinc@gmail.com

August 20, 2019

TABLE OF CONTENTS

GILMAN SPRINGS ROAD IMPROVEMENT PROJECT--ENGINEERING GEOLOGY SLOPE EVALUATION

GILMAN SPRINGS ROAD STREET IMPROVEMENT PROJECT-- ENGINEERING GEOLOGY SLOPE EVALUATION

Introduction and Scope

In accordance with our (Wilson Geosciences Inc. – WGI) discussions and communications with Diaz-Yourman & Associates (DYA), it is understood that Gilman Springs Road would be widened along the north edge necessitating cut slopes and retaining walls at various locations (DYA, 2019; see Attachment A). This technical memorandum relates to proposed modification/grading of existing cut slopes at eight (8) locations from just southeast of Bridge Street to northwest of Jack Rabbit Trail (see Google Earth image in Figure 1. The objective of the evaluation was to perform a visual field reconnaissance and a paper study in order to identify slope conditions and the potential for adverse impacts from the proposed modification/grading. The scope of work included:

1. Coordinate with Diaz-Yourman & Associates (DYA) throughout the project.
2. Collect existing readily available geologic maps and Google Earth images of the 8 locations.
3. Perform a site visit to examine, map, and photograph the 8 slopes along the north side of Gilman Springs Road at the specific proposed slope modification/grading locations shown on Figure 1 and 1" = 10' scale plans (Figure 4). Identify and describe exposed geologic conditions potentially affecting future proposed slope modification/grading.
4. Review the available data and site visit information, and analyze for potential affects of proposed slope modification/grading.
5. Prepare a technical memorandum with a brief geology discussion documenting findings and recommendations relating to proposed slope modification/grading. Respond to one set of combined DYA/Client comments.

Geologic Setting

The Project Area is located within the County of Riverside between the City of San Jacinto and the City of Moreno Valley, and occupies a portion of the Peninsular Ranges geomorphic province immediately adjacent to the Santa Jacinto Mountains on the east. The Project Area is along Gilman Springs Road and proposed modifications to existing slopes are along the north edge of the road (Figure 1). Rock outcrops in the San Jacinto Mountains hills consist of very old (66 million years and older) metamorphic and igneous "basement" bedrock, which is bordered on the west by Tertiary (>1.6 million years old)) sedimentary bedrock, Pleistocene (11,000 to 1.6 million years old) sediments, and by Holocene (0-11,000 years old) sediments on the valley floor (Morton and others, 2001; Matti and others, 2015). The distribution of geologic units within and near the Project Area is shown on Figure 2. Figure 3 provides the geologic unit names and brief descriptions of each unit.

Figure 1 shows the mapped segments (thin orange lines in Figure 1) of the San Jacinto fault zone (SJFZ) that appear to pass through or near the Project Area proposed slope modification/grading locations. This proximity to an active fault has very likely caused the rock and old alluvium to become fractured, broken, and sheared. This can make fracture and bedding planes less continuous, and possibly less susceptible to sliding failures. At the same time slopes may be more subject to sloughing and rockfall/rock rolling on the slopes.

Gilman Springs Road Widening Project Area (North Edge) Considerations

The vast majority of the proposed slope modification locations were, at the time of the visit, covered largely with thick, dry vegetation. This condition made it difficult to examine geologic exposures, and required removing small areas of brush and loose weathered materials to attempt to identify the underlying unit.

Existing Slope Characteristics

Existing slopes varied from a few feet to approximately 15 feet high and were at slope gradients with approximately 2:1 (horizontal:vertical) to vertical. Geologic materials forming the existing cut slopes consisted of very old granitic/metamorphic bedrock (symbol Klt and Figures 2 and 3) at one location, younger sedimentary bedrock (symbols Tmea, Tslt, and Tslts) at six locations, and young alluvial soils at one location. All locations contained some areas of young alluvial soils in addition to the units noted. Most slopes were largely covered by dry vegetation and showed some minor to moderate signs of surface water erosion. No gross or planar failures were noted.

At one location with intrusive igneous and metamorphic rocks, localized foliation and a few planar discontinuities were noted. The younger bedrock and alluvial units were moderately to highly weathered, and loose to medium dense. A brief summary of each of the 8 locations follows.

Existing and future cut slope stability depends on many factors related to the (a) slope configuration (height, orientation, and slope gradient), (b) alluvial or bedrock characteristics (e.g., density, continuity, and orientation of discontinuities [fractures/joints, foliation, faults], degree of weathering), and (c) ground shaking due to a potential earthquake event. It is understood that the planned slopes would be cut to slope gradients approximately between 2:1 and 1:1. With the San Jacinto fault zone passing through the Project Area, it is capable of generating substantial ground accelerations and velocities. Taken together these factors suggest caution is necessary in the design of the proposed street improvements in the Project Area.

Potential Slope Failures

Discontinuities at the granitic/metamorphic bedrock sites can have some out-of-slope (generally unfavorable = toward the street) components of dip and may be continuous over more than several feet to tens of feet, suggesting that rotational or block glide failures along fractures/joints, foliations, or faults may occur. Down-slope movement of basement rock material from the slope face to street level would most likely occur as rockfalls due to smaller wedge-type failures or local raveling of loose debris. These wedges are formed by the intersection of steeply- and relatively shallow-dipping fractures/joints that can create isolated blocks (wedges) of basement rock that could slide toward the street on discontinuities dipping at angles less shallow than a 2:1 (22.5 degree) or 1:1 (~45-degree) slope. Where metamorphic and igneous rock materials are moderately weathered and moderately to highly fractured/jointed, clay seams or clay-filling of fractures/joints can be present in the exposures near street level. Small faults can contribute to rockfall and planar failure potential.

Groundwater flowing in local fractures could reach the existing or future slope face and possibly have some lubricating affect on out-of-slope discontinuities. However, the most likely mechanism to induce a local slope failure would be due to ground shaking, possibly from local traffic, but more likely from a large nearby earthquake event. Surficial soil failures are possible where significant size/volume due to the irregular thickness and distribution across a cut slope.

Gilman Springs Road Widening Project Area Field Observations, Conclusions, and Recommendations

All observations are somewhat limited due to vegetation cover. Potential issues related to this cover are indicated below. Where reference is made to vertical or near vertical slopes these are isolated, discontinuous, and generally 1 to 3 feet high. In general it is recommended that during construction an engineering geologist conduct a field review of slopes with a gradient of 1:1 or steeper.

181+00

This slope location (proposed 1.5:1 gradient) extends between approximately stations 180+00 and 182+00. Existing slopes are less than 10 feet high and appear to be nominally with slope gradients ranging from 1:1 to 1.5:1 (see Attachment A). Based on the geology map (Figure 2) the materials are map symbols Qyf4ga (young alluvial fan deposits) and Tmea (Tertiary sandstone). Observation of the limited exposures and hand excavation of small areas near the base of the slope indicate the material is as follows:

1. Silty sand (or highly weathered sandstone), fine- to very coarse-grained, medium brown, loose to medium dense, with subangular gravel to 2-inches in diameter, and subject to surface water erosion.

No signs of gross instability were observed and exposures or hand-excavated areas are believed to be to reach reasonable conclusions. We saw no indication of the Tmea and believe the materials were entirely Qyf4ga. Slopes with gradients of 1:1 or flatter should result in grossly stable excavations. New slopes may be subject to water erosion depending upon upslope drainage patterns.

Further geologic investigation is not considered necessary. Clearing vegetation to gain a full view of the entire slope area could be a helpful option, but would not be mandatory based on our field observations.

205+00

This slope location (proposed 1:1 gradient) extends between approximately stations 204+50 and 206+00. Existing slopes are less than 10 feet high and appear to be nominally with slope gradients from 2:1 to vertical (see Attachment A). Based on the geology map (Figure 2) the materials are map symbols Qyf3ga (young alluvial fan deposits), Tmea (Tertiary sandstone), and Klt (Cretaceous granitic and metamorphic bedrock). Observation of the limited exposures and hand excavation of small areas near the base of the slope indicate the material is as follows (where station numbers overlap the younger unit overlies the older unit):

1. Silty sand (or moderately weathered sandstone), fine- to very coarse-grained, medium brown, medium dense to dense, with gravel to 2-inches in diameter, and somewhat subject to surface water erosion. (205 to 206 and 204+50 to 204+75)

2. Silty clayey sand (or moderately weathered sandstone), fine- to very coarse-grained, dark reddish brown, dense to very dense, with minor gravel, and minor susceptibility to surface water erosion; limited to an exposure of approximately 20 to 30 feet within the bedrock described below. (204+70 to 205)

3. Granitic and metamorphic bedrock, moderate to very hard, slightly to moderately weather, medium to very coarse-grained, evidence of shearing, some apparently discontinuous planar features (joints/fractures). (204+50 to 205+50)

No signs of gross instability were observed and exposures or hand-excavated areas are believed to be to reach reasonable conclusions. While we saw no indication of a sandstone material, Tmea may be present as the unit 2 above. We believe most of the slope to be alluvial materials Qyf3ga. Slopes with gradients of 1:1 or flatter should result in grossly stable excavations. New slopes may be subject to water erosion depending in alluvial areas depending upon upslope drainage patterns. Excavation of the Klt should be moderately difficult with standard heavy duty excavators.

Further geologic investigation is not considered necessary due to low proposed cut slope height. But it may be advisable to clean the areas of Klt and determine if any continuous adversely oriented failure planes are present. Clearing vegetation to gain a full view of the entire slope area could be a helpful option, but would not be mandatory based on our field observations.

228+50

No site plan/contour map was available for this location. This slope location (proposed 1:1 gradient) extends for approximately 170 feet, between stations 227+00 and 230+00. Existing slopes are less than 15 to 20 feet high and appear to be nominally with slope gradients of greater than 1:1. Based on the geology map (Figure 2) the materials are map symbols Qyf3ga (young alluvial fan deposits) and Klt (Cretaceous granitic and metamorphic bedrock). Observation of the exposures and hand excavation of small areas near the base of the slope indicate the material is as follows:

1. Silty sand (or moderately weathered sandstone), fine- to very coarse-grained, medium brown, medium dense to dense, with gravel to 2-inches in diameter, scattered small boulders and cobbles, and somewhat subject to surface water erosion.

A recent cut slope was created just to the north of 228+50 and is in both Klt and Qyf3ga. No Klt was observed at the subject location, although it may be buried at a shallow depth beneath the Qyf3ga. There were no signs of gross instability observed and exposures or small hand-excavated areas are believed to be to reach reasonable conclusions. Slopes with gradients of 1:1

or flatter should result in grossly stable excavations. New slopes may be subject to water erosion depending in alluvial areas depending upon upslope drainage patterns. Should it be encountered, excavation of the Klt should be moderately difficult with standard heavy duty excavators.

Further geologic investigation is not considered necessary due to low proposed cut slope height. Clearing vegetation to gain a full view of the entire slope area could be a helpful option, but would not be mandatory based on our field observations.

236+00

This slope location (proposed 1.5:1 gradient) extends between approximately stations 235+00 and 237+00. Existing slopes are less than 10 feet high and appear to be nominally with slope gradients of 2:1, 1:1, and vertical (see Attachment A). Based on the geology map (Figure 2) the materials are map symbols Qyfa (younger alluvial fan deposits) and Tslt (Tertiary sandstone). Observation of the limited exposures and hand excavation of small areas near the base of the slope indicate the material is as follows:

1. Silty sand (or very weathered sandstone), fine- to very coarse-grained, medium brown, medium dense to dense, with gravel to 2-inches in diameter, scattered small boulders and cobbles, and somewhat subject to surface water erosion.

No signs of gross instability were observed and exposures or hand-excavated areas are believed to be to reach reasonable conclusions. We expect the presence of Tslt at the base of the slope and believe it is overlain by the alluvial materials Qyfa. Slopes with gradients of 1:1 or flatter should result in grossly stable excavations. New slopes may be subject to water erosion depending in alluvial areas depending upon upslope drainage patterns.

Further geologic investigation is not considered necessary due to low proposed cut slope height. Clearing vegetation to gain a full view of the entire slope area could be a helpful option, but would not be mandatory based on our field observations.

336+50

This slope location (proposed 1.5:1 gradient) location extends between approximately stations 336+20 and 337+20. Existing slopes are less than 5 feet high and appear to be nominally with slope gradients of 2:1 and greater than 1:1 (see Attachment A). Based on the geology map (Figure 2) the materials are map symbols Qyf3 (younger alluvial fan deposits) and Qyls (younger landslide deposits involving Tslts, Tertiary sandstone). Observation of the limited exposures and hand excavation of small areas near the base of the slope indicate the material is as follows:

1. Silty sand and sandy silt (or very weathered sandstone and siltstone), fine- to very coarse-grained, medium brown, medium dense to dense, with gravel to 2-inches in diameter, scattered small boulders and cobbles, and somewhat subject to surface water erosion.
2. Silty sand (or weathered sandstone), fine- to very coarse-grained, medium brown, medium dense to very dense, with abundant gravel, scattered small boulders and cobbles, and somewhat subject to surface water erosion.

No signs of gross instability were observed and exposures or hand-excavated areas are believed to be to reach reasonable conclusions. The Qyls is likely stabilized and its stability should be unaffected with the relatively minor cut slope proposed. We expect the presence of Tslts as a part of the Qyls and believe it is overlain in part by the alluvial materials Qyf3. Slopes with gradients of 1:1 or flatter should result in grossly stable excavations. New slopes may be subject to water erosion depending in alluvial areas depending upon upslope drainage patterns.

Further geologic investigation is not considered necessary due to low proposed cut slope height. Clearing vegetation to gain a full view of the entire slope area could be a helpful option, but would not be mandatory based on our field observations.

357+00

This slope location (proposed 1.5:1 gradient) extends between approximately stations 357+25 and 356+00. Existing slopes are between 10 to 12 feet high and appear to be with slope gradients of approximately 1:1 (see Attachment A). Based on the geology map (Figure 2) the materials are map symbols Qyf3 (younger alluvial fan deposits) and Qols (older landslide deposits possibly involving Tslts, Tertiary sandstone). These two units could not be differentiated at this location. Observation of the limited exposures and hand excavation of small areas near the base of the slope indicate the material is as follows:

1. Silty sand (or weathered sandstone), fine- to very coarse-grained, medium brown, medium dense to very dense, with abundant gravel, scattered small boulders and cobbles, and somewhat subject to surface water erosion. The percentage of gravel, cobbles, and small boulders appears to increase to the northwest.

No signs of gross instability were observed and exposures or hand-excavated areas are believed to be to reach reasonable conclusions. The Qyls is likely stabilized and its stability should be unaffected with the relatively minor cut slope proposed. We expect the presence of Tslts as a part of the Qyls and believe it is overlain in part by the alluvial materials Qyf3. Slopes with gradients of 1:1 or flatter should result in grossly stable excavations. New slopes may be subject to water erosion depending in alluvial areas depending upon upslope drainage patterns.

Further geologic investigation is not considered necessary due to low proposed cut slope height. Clearing vegetation to gain a full view of the entire slope area could be a helpful option, but would not be mandatory based on our field observations.

358+00

This slope location (proposed 1.5:1 gradient) extends between approximately stations 357+25 and 358+40 (see Attachment A). Existing slopes are between 10 to 12 feet high and with slope gradients of approximately 1:1. Based on the geology map (Figure 2) the materials are map symbols Qyf3 (younger alluvial fan deposits) and Qols (older landslide deposits possibly involving Tslts, Tertiary sandstone). These two units could not be differentiated at this location. Observation of the limited exposures and hand excavation of small areas near the base of the slope indicate the material is as follows:

1. Silty sand (or weathered sandstone), fine- to very coarse-grained, medium brown, medium dense to very dense, with abundant gravel, scattered small boulders and cobbles, and somewhat subject to surface water erosion. The percentage of gravel, cobbles, and small boulders appears to significantly increase to the northwest.

No signs of gross instability were observed and exposures or hand-excavated areas are believed to be to reach reasonable conclusions. The Qyls is likely stabilized and its stability should be unaffected with the relatively minor cut slope proposed. We expect the presence of Tslts as a part of the Qyls and believe it is overlain in part by the alluvial materials Qyf3. Slopes with gradients of 1:1 or flatter should result in grossly stable excavations. New slopes may be subject to water erosion depending in alluvial areas depending upon upslope drainage patterns.

Further geologic work is not considered mandatory due to the relatively low proposed cut slope height. Clearing vegetation to gain a full view of the entire slope area could be a helpful option, but would not be mandatory based on our field observations.

360+00

This slope location (proposed 1.5:1 gradient) extends between approximately stations 359+00 and 361+00 (see Attachment A). Existing slopes are between 10 to 12 feet high and with slope gradients that appear to be approximately 1:1. Based on the geology map (Figure 2) the materials are map symbols Qvyfu (very young alluvium), Qyf3 (younger alluvial fan deposits) and Qols (older landslide deposits possibly involving Tslts, Tertiary sandstone). Qvyfu was not observed and the two older units could not be differentiated at this location. Observation of the limited exposures and hand excavation of small areas near the base of the slope indicate the material is as follows:

1. Silty sand (or weathered sandstone), fine- to very coarse-grained, medium brown, dense to very dense, with abundant gravel, scattered small boulders and cobbles, and somewhat subject to surface water erosion. The percentage of gravel, cobbles, and small boulders appears to significantly increase to the northwest.

No signs of gross instability were observed and exposures or hand-excavated areas are believed to be to reach reasonable conclusions. The Qyls is likely stabilized and its stability should be unaffected with the relatively minor cut slope proposed. We expect the presence of Tslts as a part of the Qyls and believe it is overlain in part by the alluvial materials Qyf3. Slopes with gradients of 1:1 or flatter should result in grossly stable excavations. New slopes may be subject to water erosion depending in alluvial areas depending upon upslope drainage patterns.

Further geologic investigation is not considered necessary due to the relatively low proposed cut slope height. Clearing vegetation to gain a full view of the entire slope area could be a helpful option, but would not be mandatory based on our field observations.

Limitations

Engineering geology is characterized by uncertainty, and is often described as an inexact science. Opinions presented herein are based on the evaluation of surface exposures that may not be fully representative of conditions within the slope face where the excavation would ultimately be made. Our evaluation used past experience and professional judgment to evaluate this data. No subsurface investigation or clearing of slope vegetation was authorized or performed, and conclusions, recommendations, and other information contained in this report are based upon the assumption that the conditions observed are representative of the materials and discontinuities that would be found in the final excavation area. Quantitative slope stability analysis is the responsibility of others. It must be recognized that variations from these conditions can occur.

The opinions presented should be considered "advice." Other consultants could arrive at different opinions using the same information. These opinions have been presented after consideration of the data reviewed and are believed to comply with normal industry standards. Although some risk will always remain, lower risk of future geologic hazard related problems would usually result if cut slope configurations avoid known or suspected hazards, if quantitative slope stability analysis is performed, and if conservative design criteria were adopted for the future cut slope. Final decisions on matters presented are the responsibility of the client and/or the governing agencies. Wilson Geosciences Inc. and its employees make no warranties in any respect as to the engineering geologic performance of the site.

References Cited

DYA, 2019, Site plans (40-scale) and cross-section profiles for the proposed street improvement Project Area, and Google Earth images for the overall Project Area and each proposed cut slope location.

Morton, D.M., Matti, J.C., Alvarez, Rachel, and Cossette, P.M., 2001, Geologic map of the Lakeview 7.5' quadrangle, Riverside County, California: U.S. Geological Survey, Open-File Report OF-2001-174, scale 1:24,000.

Matti, J.C., Morton, D.M., and Langenheim, V.E., 2015, Geologic and geophysical maps of the El Casco 7.5' quadrangle, Riverside County, southern California, with accompanying geologic-map database: U.S. Geological Survey, Open-File Report OF-2010-1274, scale 1:24,000.

If you have any questions about this report, please contact the undersigned at your convenience.

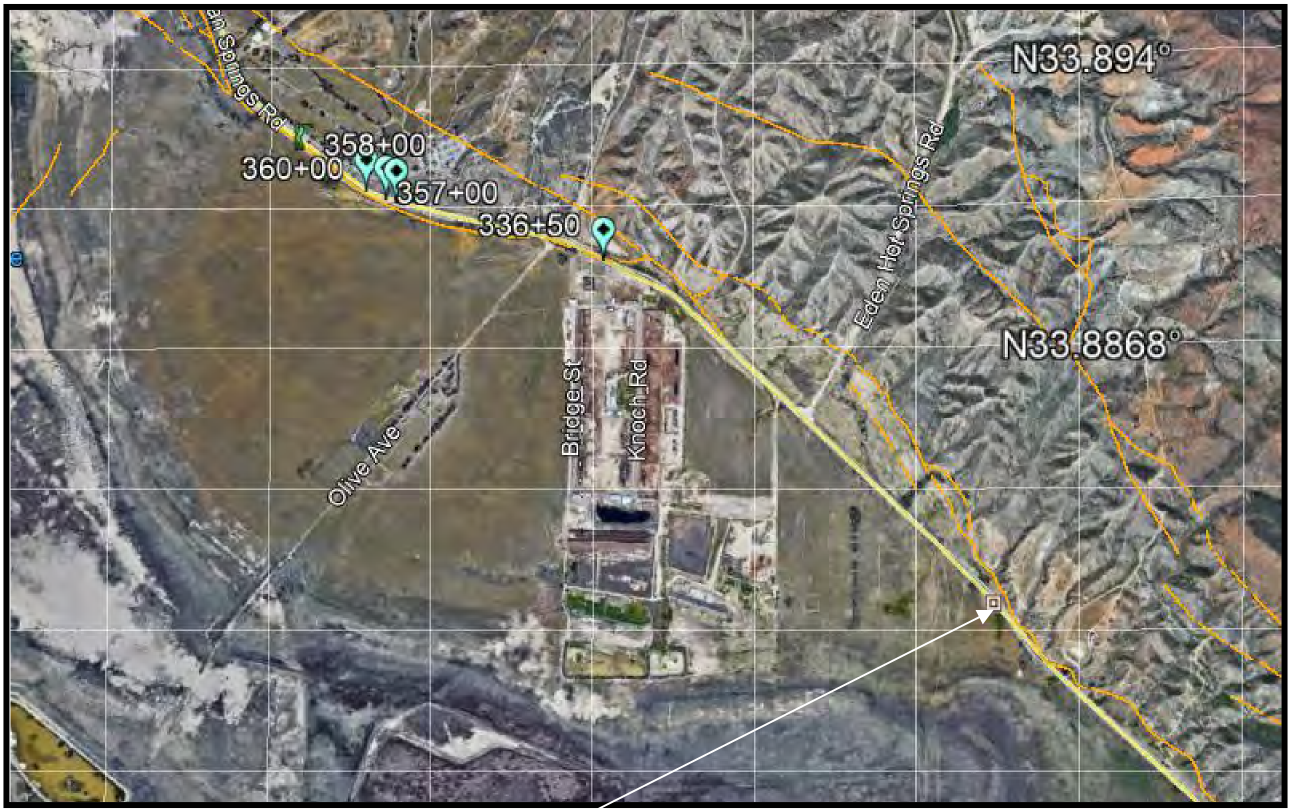
Sincerely,
WILSON GEOSCIENCES INC.

Kenneth Wilson
Principal Geologist
P.G. #3175, C.E.G. #928

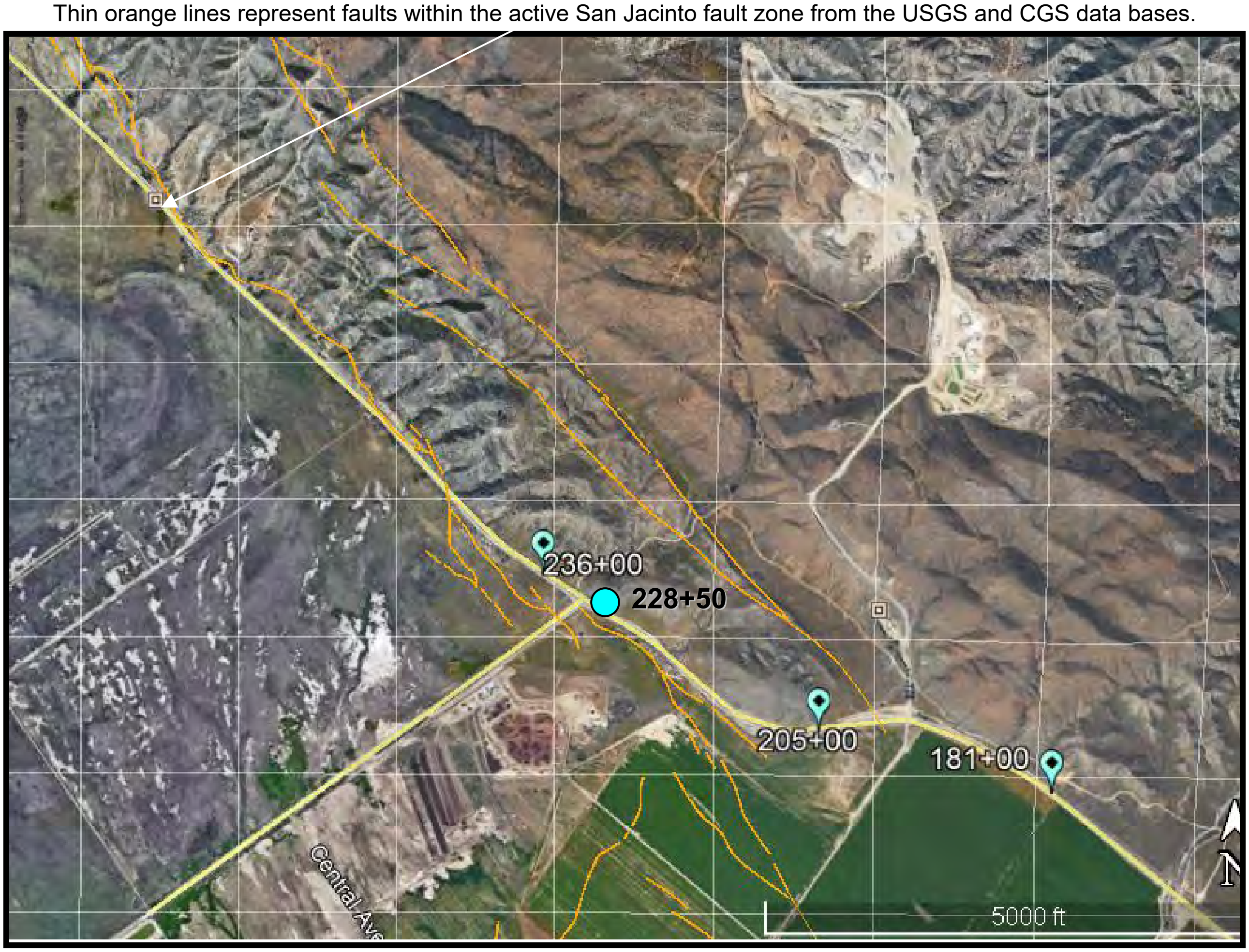
PROFESSIONAL GEOLOGIST
KENNETH LEE WILSON
No. 928
CERTIFIED
ENGINEERING
GEOLOGIST
STATE OF CALIFORNIA

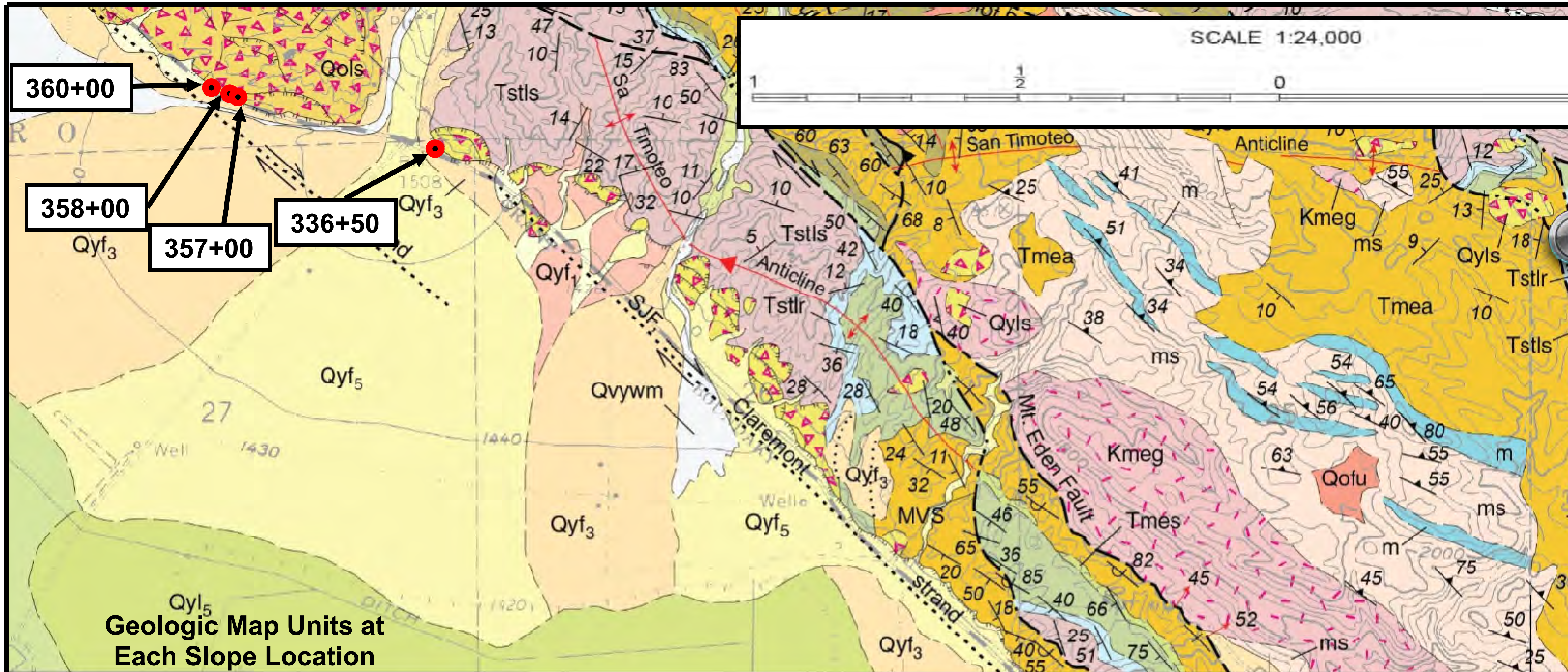
Figures (Pages Follow)

FIGURE 1 - Proposed Cut Slope Location Map, Gilman Springs Road, Riverside County, California



Thin orange lines represent faults within the active San Jacinto fault zone from the USGS and CGS data bases.





NORTH

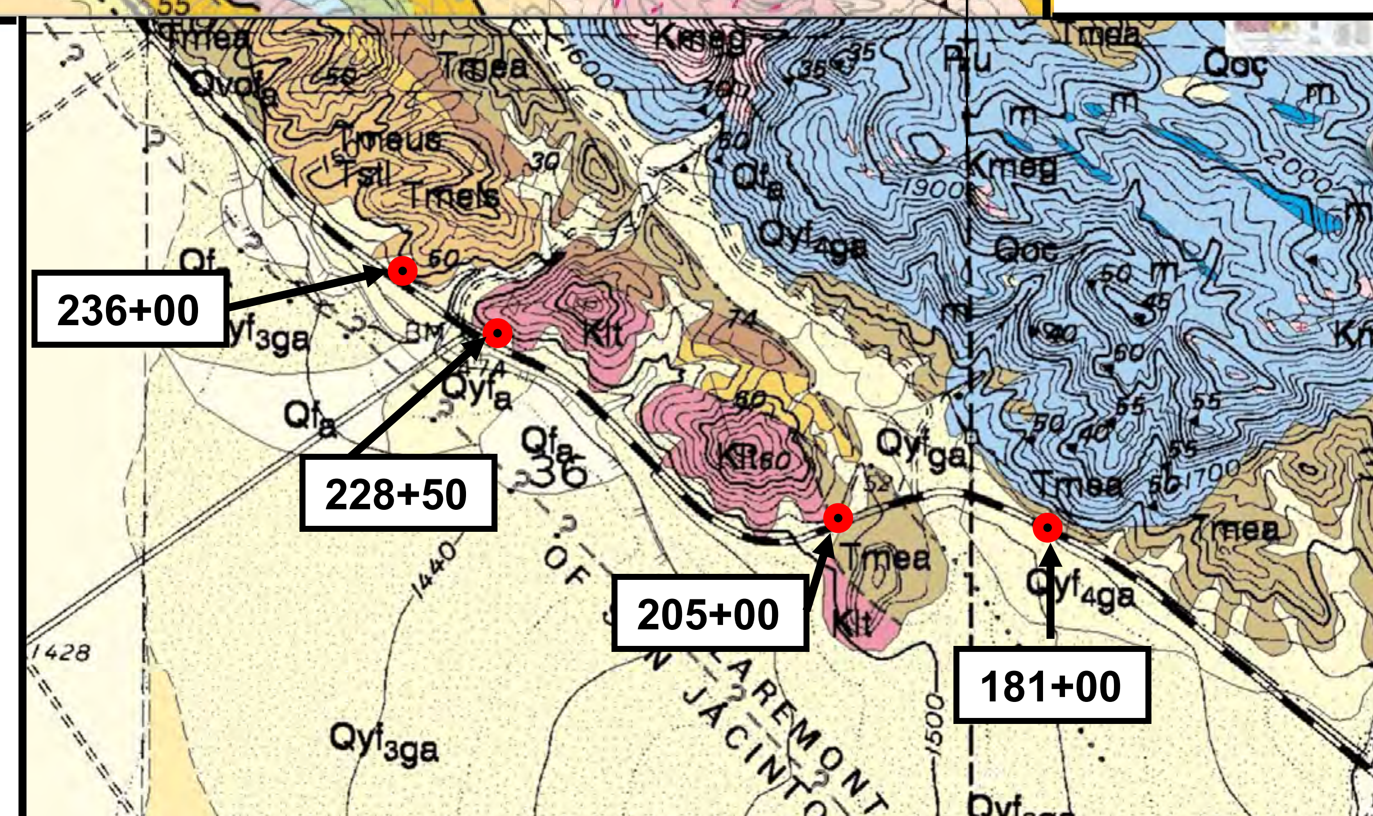
Map Scale 1-inch ≈ 1200-feet

Matti, J.C., Morton, D.M., and Langenheim, V.E., 2015, Geologic and geophysical maps of the El Casco 7.5' quadrangle, Riverside County, southern California, with accompanying geologic-map database: U.S. Geological Survey, Open-File Report OF-2010-1274, scale 1:24,000.

181+00 – Qyf4ga and Tmea
205+00 - Qyf3ga, Tmea, and Klt
228+50 – Qyf3ga and Klt
236+00 – Qyfa and Tstl
336+50 – Qyls, Qyf3, and Tstls
357+00 - Qyf3, Qols and Tstls
358+00 - Qyf3, Qols and Tstls
360+00 – Qvyfu, Qyf3, Qols and Tstls

See Figure 3 for Geologic Unit Descriptions

FIGURE 2 - Geologic Map, Proposed Cut Slope Locations, and Geologic Units Designated at Each Location, Gilman Springs Road, Riverside County, California



Morton, D.M., Matti, J.C., Alvarez, Rachel, and Cossette, P.M., 2001, Geologic map of the Lakeview 7.5' quadrangle, Riverside County, California: U.S. Geological Survey, Open-File Report OF-2001-174, scale 1:24,000.

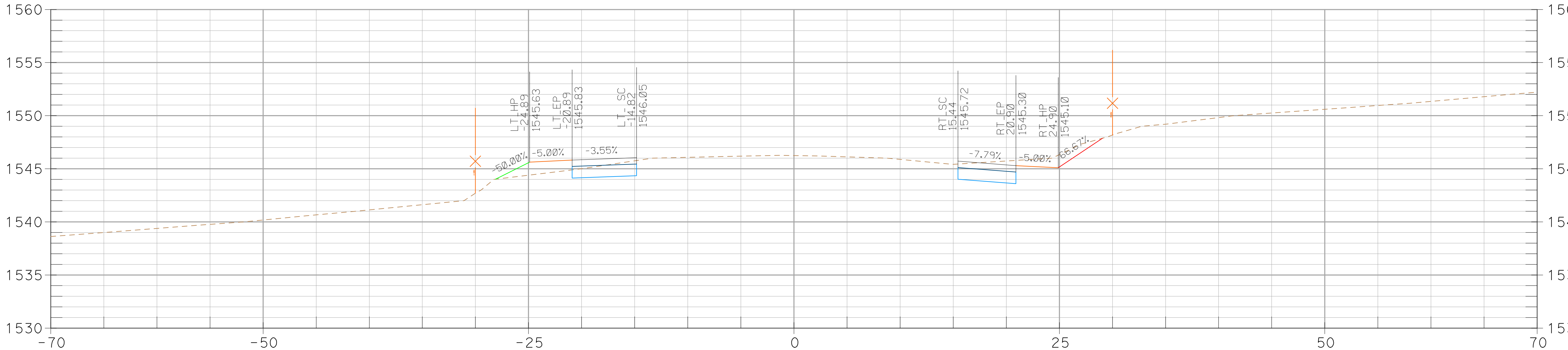
FIGURE 3 - Geologic Map Units Designated on Figure 2 at Each Proposed Cut Slope Location, Gilman Springs Road, Riverside County, California

Unit	Description
Qvyfu	**Very young alluvial-fan deposits, undifferentiated (uppermost Holocene)**—Very slightly to slightly consolidated, undissected deposits of sandy and gravelly sediment that form actively sedimented parts of alluvial fans; occurs only locally
Qyf3	**Young alluvial-fan deposits, unit 3 (middle Holocene)** **Young alluvial-fan deposits (Holocene & uppermost Pleistocene)**—Slightly to moderately consolidated sandy, gravelly, and muddy sediment deposited by streams flowing on alluvial-fan landforms. Units distinguished on the basis of soil-profile development and relative position in local terrace-riser succession.
Qyf4	**Young alluvial-fan deposits, Unit 4 (Holocene and late Pleistocene)**—Forms slightly dissected deposits downstream from canyon mouths in northeastern part of quadrangle. Consists of sandy, cobble-boulder, gravel and poorly sorted gravelly sand that is light gray
Qyf3	**Young alluvial-fan deposits, Unit 3 (Holocene and late Pleistocene)**—Forms remnant terrace risers downstream from canyon mouths. Consists of sandy, cobble-boulder, gravel and poorly sorted gravelly sand that is light gray
Qyls	**Young landslide deposits (Holocene & uppermost Pleistocene)**—Slightly dissected slope-movement deposits. Locally may include old landslide material
Qols	**Old landslide deposits (upper to middle Pleistocene)**—Moderately dissected slope-movement deposits. Probably inactive under current climatic and tectonic conditions
Tstl	**San Timoteo beds of Frick (1921) (Pliocene)**—Mostly gray, well sorted, fine-grained sandstone. Contains subordinate pebble lenses in which all lithic clasts are derived from Transverse Ranges province rocks. Some medium grained sandstone interbeds. Unit represents distal flood plain type of deposit. Only lowest member of formation found in Lakeview quadrangle
Tstls	**Sandstone unit (lower Pliocene)**—Consists dominantly of sandy rock interbedded with sparse conglomeratic rock; mudrock minor, except in stratigraphic sequence west of Moreno Valley strand of San Jacinto Fault at west edge of quadrangle. Rock is well consolidated to lithified, with some intervals cemented by calcareous and (or) siliceous cement. Contact with overlying middle member (unit Tstm) is transitional across stratigraphic interval as much as 50 m thick
Tmea	**Arkosic sandstone member (Miocene)**—Well indurated, gray-brown and light-gray, arkosic, and lithic sandstone. Sandstone is massively, crudely bedded and thick bedded. Contains clasts derived from Peninsular Ranges province rocks. Has limy concretions, but not nearly as common as in overlying lower sandstone member (Tmels). Referred to as Potrero arkose by Frick (1921). Conformably overlies conglomerate member (Tmec). Locally contains:
Klt	**Tonalite of Laborde Canyon (Cretaceous)**—Biotite-hornblende tonalite. Relatively dark, foliated; contains abundant hornblende and thin, small mesocratic inclusions. Medium-grained; average color index 15. Thoroughly fractured and deeply weathered. Forms small, discontinuously and poorly exposed body in northeast part of Lakeview quadrangle adjacent to Claremont Fault, west of, but not in, Laborde Canyon

ATTACHMENT A

DYA, 2019, Site Plans (40-Scale) and Cross-Section Profiles for the Proposed Street Improvement Project Area


BEGIN PHASE 4A 178+00
LAUDA FAMILY LIMITED PARTNERSHIP APN 423-240-010
SO. CALIF. EDISON APN 423-240-014
LAUDA FAMILY LTD PARTNERSHIP APN 423-240-025
SEE MATCH LINE BELOW 195+50 "G" LINE
EXIST R = 2854.34'
GILMAN SPRINGS RD
EXIST R = 1312.34'
(SLOPE)
WESTERN RIVERSIDE COUNTY REGIONAL CONSERVATION AUTHORITY APN 423-240-012
STA 180+00 BEG SLOPE TRANS
STA 180+50 END SLOPE TRANS BEG 1.5:1 SLOPE
STA 181+50 END 1.5:1 SLOPE BEG SLOPE TRANS
STA 182+00 END SLOPE TRANS
SO. CALIF. EDISON APN 423-240-015
WESTERN RIVERSIDE COUNTY REGIONAL CONSERVATION AUTHORITY APN 424-190-012 (NEW) (PREVIOUSLY APN 421-200-006)
WESTERN RIVERSIDE COUNTY REGIONAL CONSERVATION AUTHORITY APN 423-240-005
LEGEND
EXISTING R/W
EXISTING EASMENT (TYPE)
EP
HINGE LINE
GRADING CUT LIMIT
GRADING FILL LIMIT
SLOPE TRANSITION
PROPOSED 1.5 :1 CUT SLOPE
PROPOSED 1 :1 CUT SLOPE
LAUDA FAMILY LTD PARTNERSHIP APN 423-240-025
LAUDA FAMILY LTD PARTNERSHIP APN 423-240-026
SEE MATCH LINE ABOVE 195+50 "G" LINE
CHANDLER AGGREGATE
EXIST R = 1312.34'
GILMAN SPRINGS RD
EXIST R = 1312.34'
WESTERN RIVERSIDE COUNTY REGIONAL CONSERVATION AUTHORITY APN 423-240-005
STA 204+50 BEG SLOPE TRANS
STA 205+50 END SLOPE TRANS BEG 1:1 SLOPE
STA 206+00 END 1:1 SLOPE
EXIST CUT SLOPE= APROX 1:1 (SEE CROSS SECTIONS)
(SLOPE)
STA 211+00 BEG SLOPE TRANS
STA 211+50 END SLOPE TRANS BEG 1:1 SLOPE
SEE MATCH LINE SHEET 2 212+50 "G" LINE
WESTERN RIVERSIDE COUNTY REGIONAL CONSERVATION AUTHORITY APN 423-240-027
0 40 80 120
SCALE: 1" = 40'
ENGINEERING COMPANY
NCM ENGINEERING CORP
4740 GREEN RIVER ROAD
SUITE 218
CORONA, CA 92880
PREPARED BY:
R.C.E. NO.
DATE
GILMAN SPRINGS RD WIDENING
SHEET 1 OF 7



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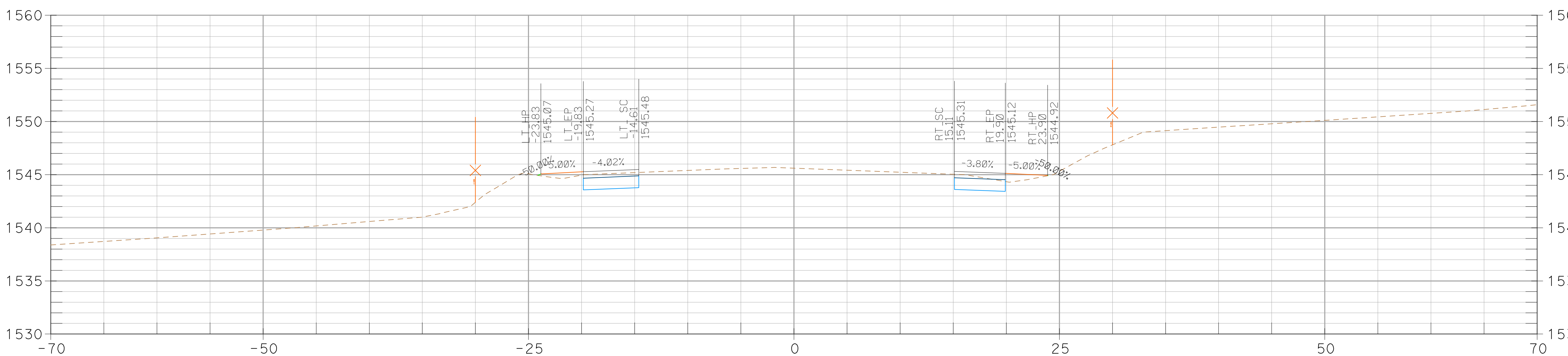

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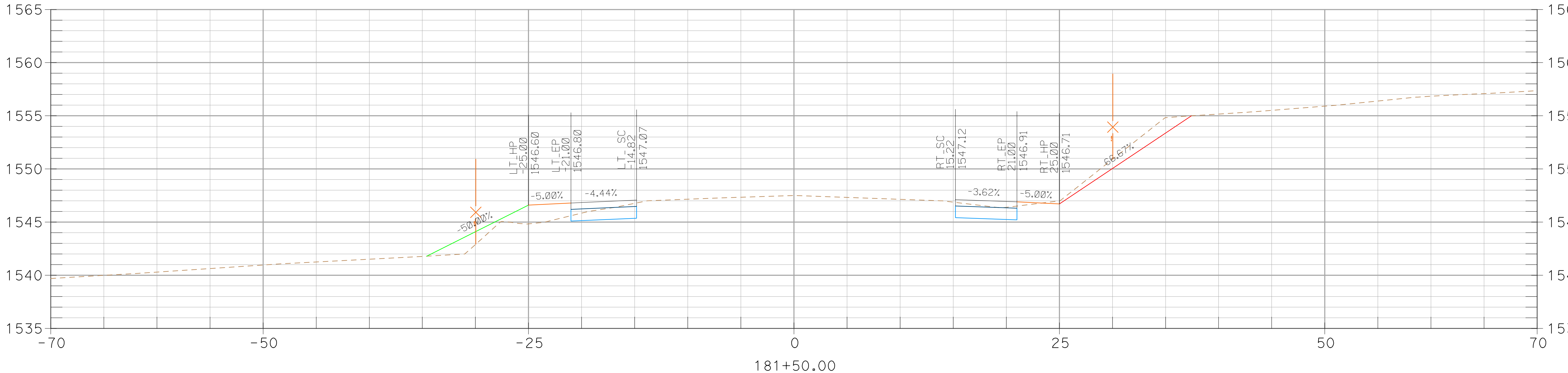
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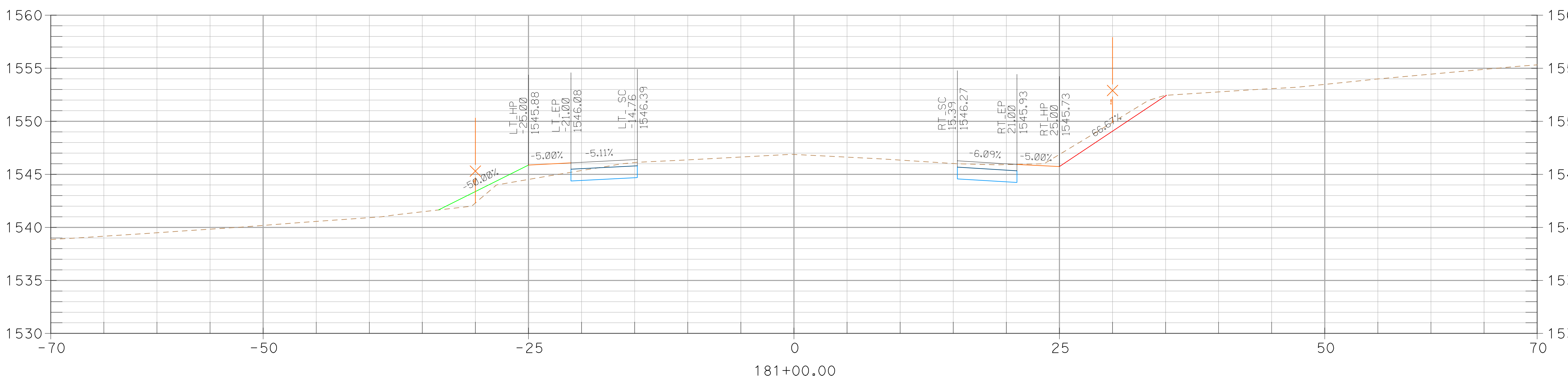
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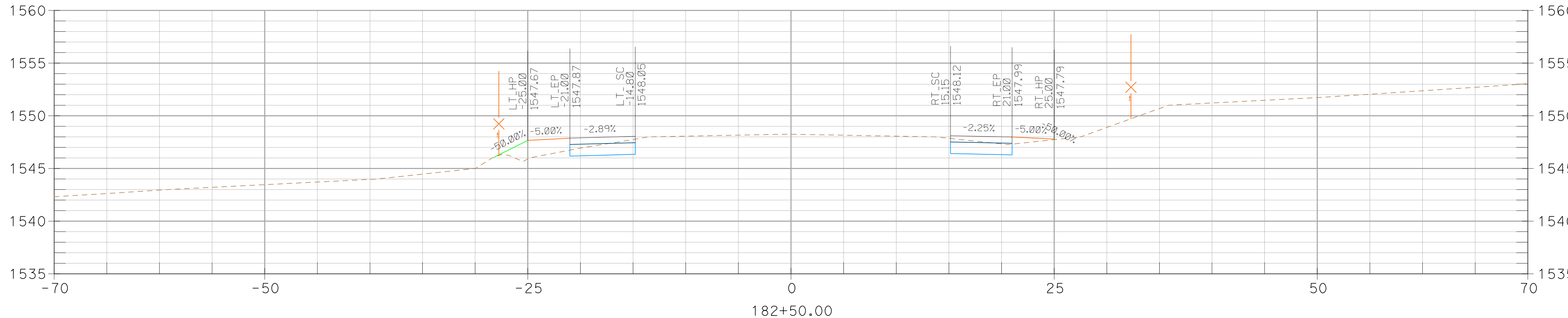
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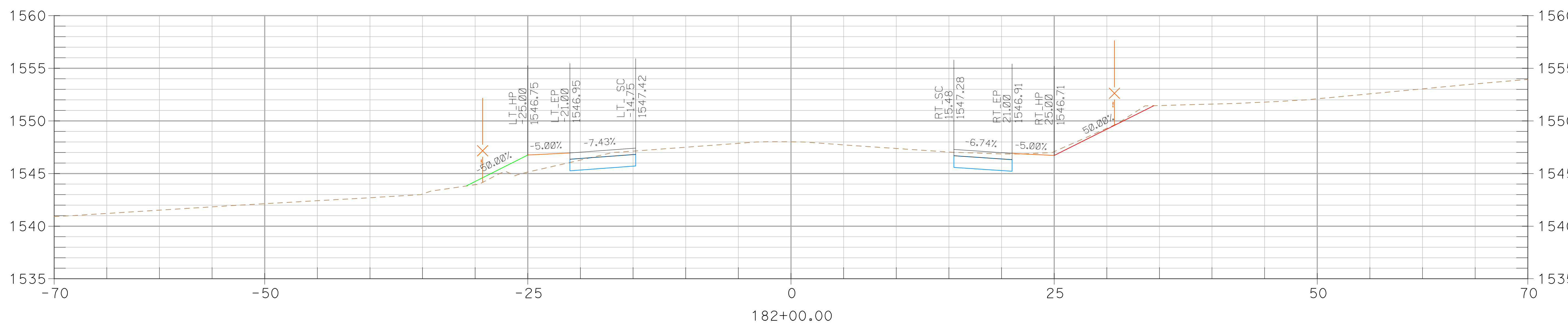
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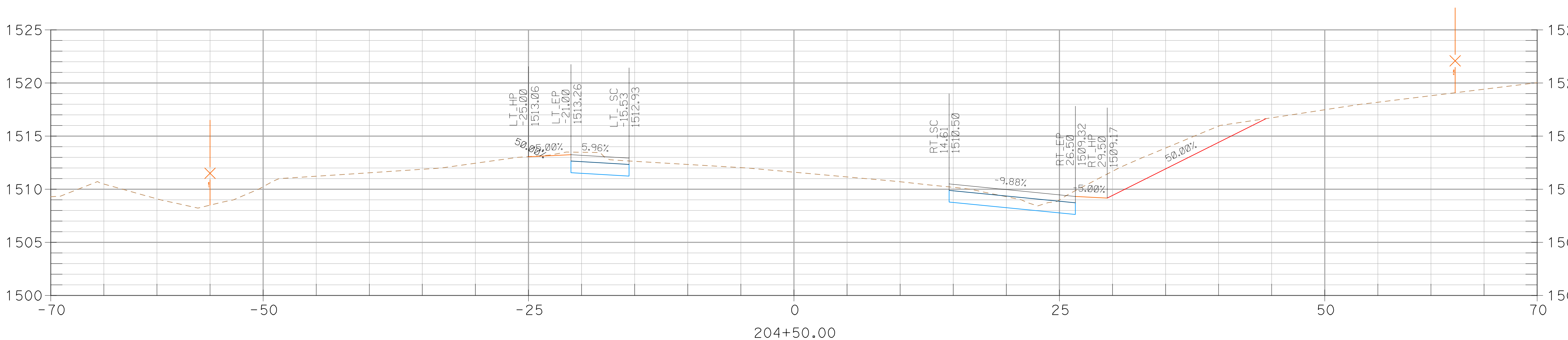
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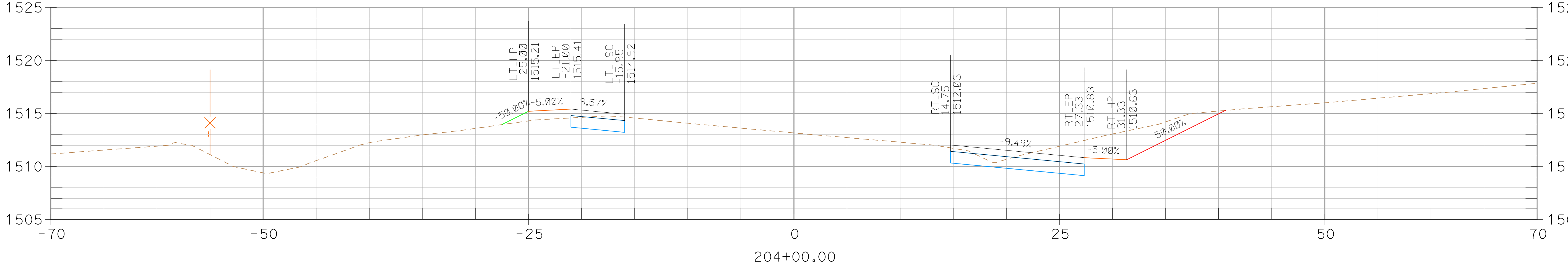
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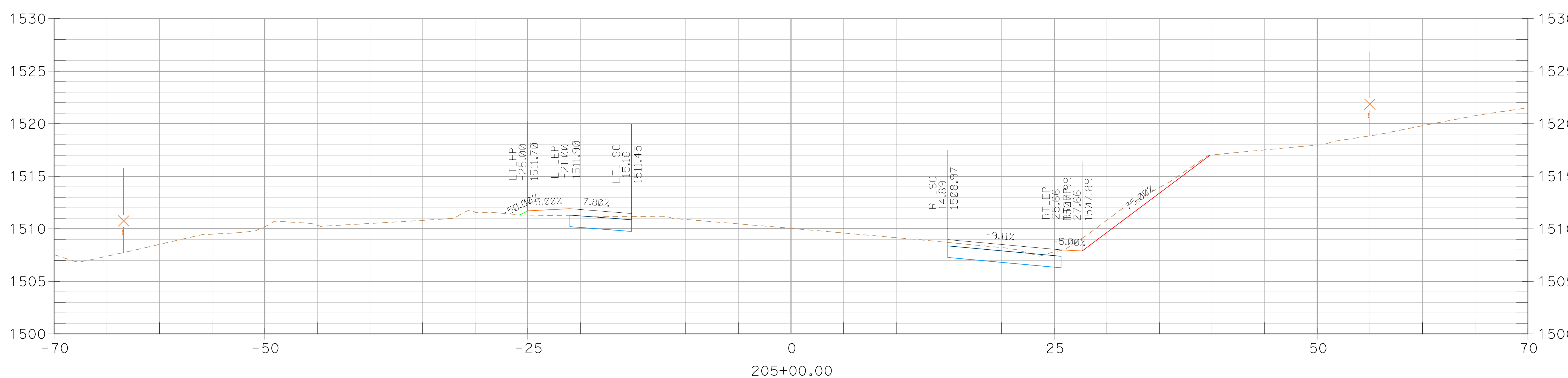
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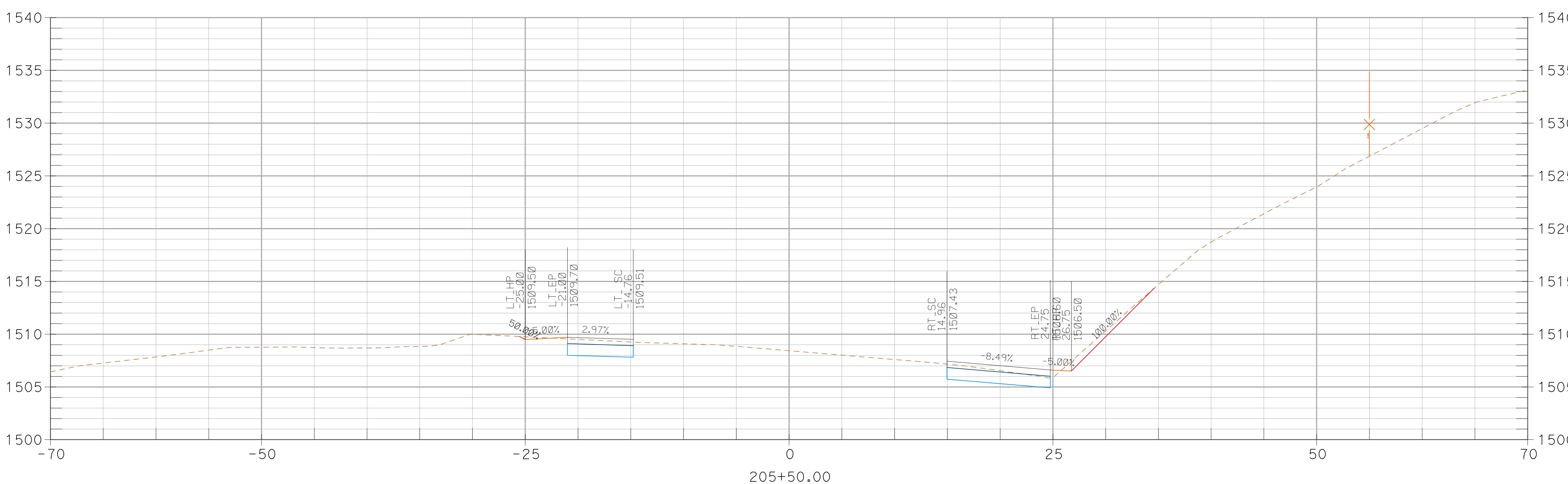
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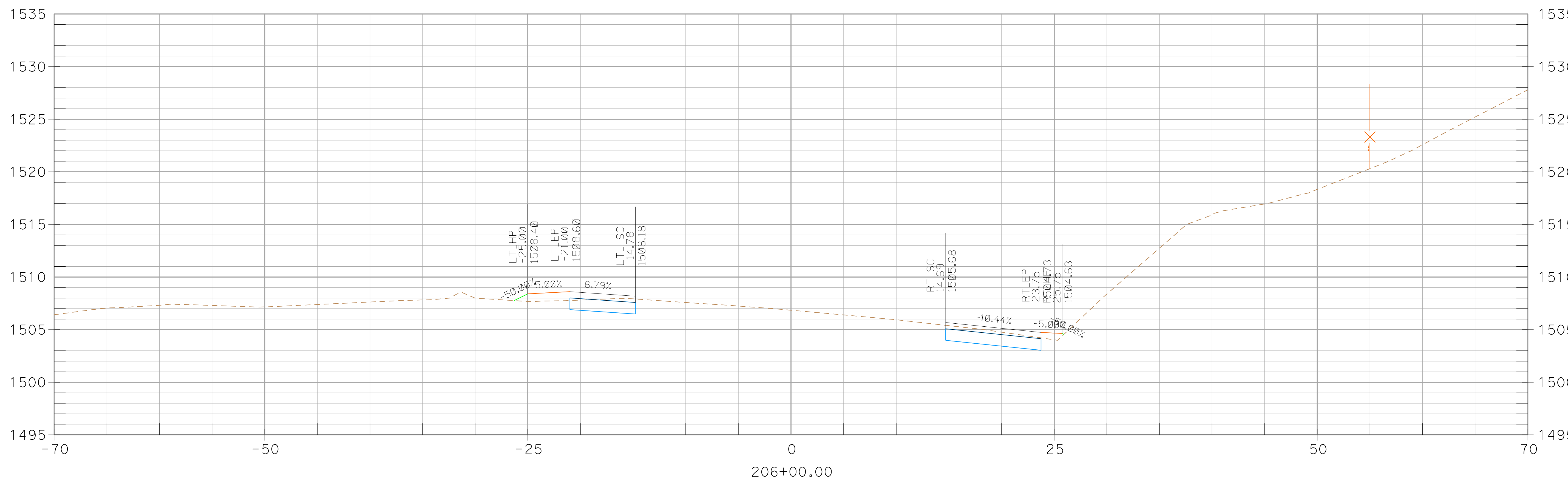
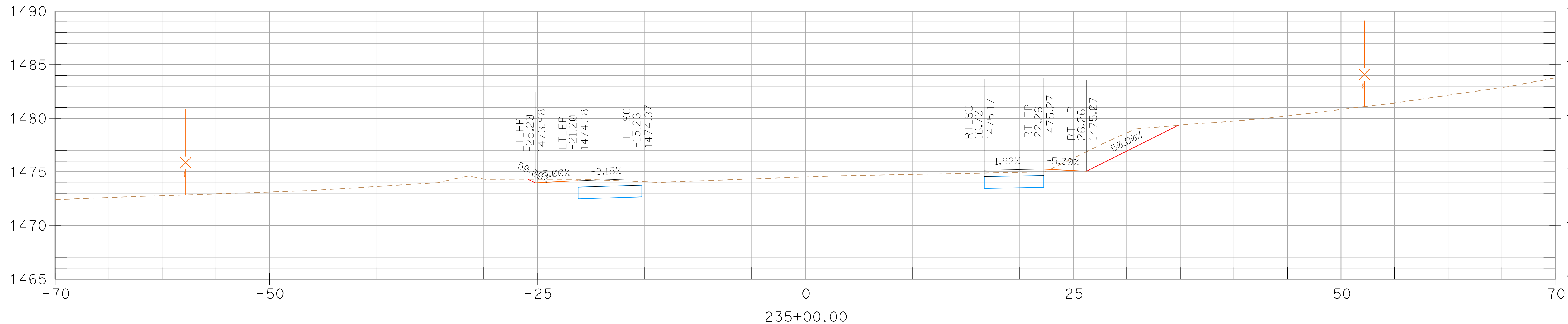
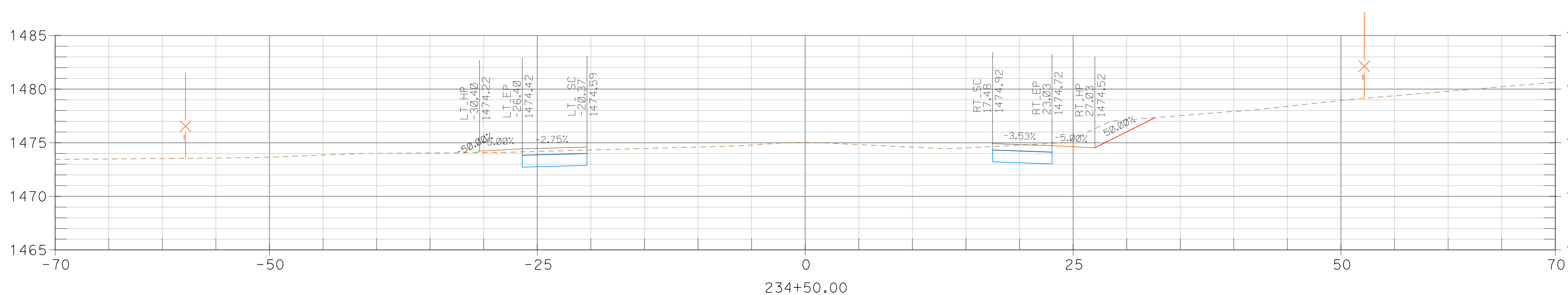
-21.00
1509.70
LT_SC
-14.76
1509.51
50.00%
5.00%
2.97%
RT_SC
14.96
1507.43
-8.49%
RT_EP
24.75
-5.00%
26.75
1506.50
100.00%
1540
1535
1530
1525
1520
1515
1510
1505
1500
-70
-50
-25
0
25
50
70
205+50.00


1535
1530
1525
1520
1515
1510
1505
1500
1495
LT_HP
-25.00
1508.40
5.00%
LT_EP
-21.00
1508.60
6.79%
LT_SC
-14.78
1508.18
RT_SC
14.69
1505.68
-10.44%
RT_EP
23.75
25.75
1504.63
-5.00%
-70
-50
-25
0
25
50
70
206+00.00

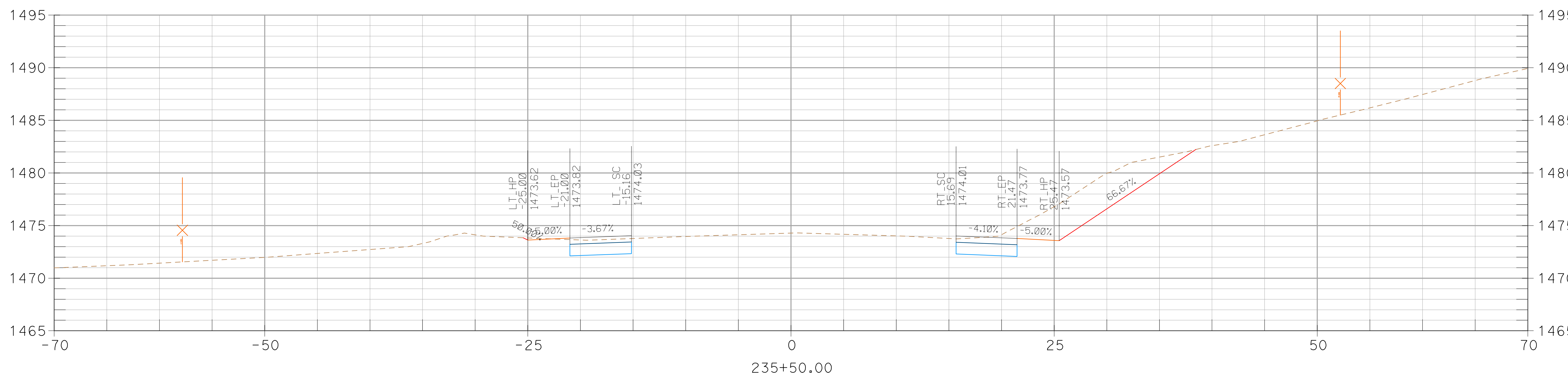
LAUDA FAMILY LTD PARTNERSHIP
APN 423-240-026
FRANCISCO & RUFINA RAMIREZ
APN 425-080-050
SEE MATCH LINE SHEET 1
212+50 "G" LINE
EXIST R = 1312.34'
GILMAN SPRINGS RD
EXIST R = 1312.34'
EXIST R = 2854.34'
SEE MATCH LINE BELOW
229+50 "G" LINE
(SLOPE)
STA 213+50
END 1:1 SLOPE
BEG SLOPE TRANS
STA 214+50
END SLOPE TRANS
THIS CUT SLOPE IS LESS THAN 3' TALL
WESTERN RIVERSIDE COUNTY
REGIONAL CONSERVATION AUTHORITY
APN 423-240-027
STA 228+50
BEG SLOPE TRANS
STA 229+50
END SLOPE TRANS
BEG 1:1 SLOPE
FRANCISCO & RUFINA RAMIREZ
APN 425-080-050
BRIDGE ST
STATE OF CALIFORNIA
WILDLIFE CONSERVATION BOARD
APN 423-240-002
STATE OF CALIFORNIA
WILDLIFE CONSERVATION BOARD
APN 423-230-010
SEE MATCH LINE ABOVE
229+50 "G" LINE
EXIST R = 2854.34'
GILMAN SPRINGS RD
EXIST R = 2854.34'
SEE MATCH LINE SHEET 3
246+50 "G" LINE
(SLOPE)
1:1 CUT SLOPE HAS ALREADY
BEEN CONSTRUCTED HERE
AS PART OF THE BRIDGE ST
SIGNAL PROJECT
STA 231+50
END 1:1 SLOPE
BEG SLOPE TRANS
STA 232+50
END SLOPE TRANS
STA 235+00
BEG SLOPE TRANS
STA 235+50
END SLOPE TRANS
BEG 1.5:1 SLOPE
STA 236+50
END 1.5:1 SLOPE
BEG SLOPE TRANS
STA 237+00
END SLOPE TRANS
LEGEND
EXISTING R/W
EXISTING EASMENT (TYPE)
EP
HINGE LINE
GRADING CUT LIMIT
GRADING FILL LIMIT
SLOPE TRANSITION
PROPOSED 1.5 :1 CUT SLOPE
PROPOSED 1 :1 CUT SLOPE
WESTERN RIVERSIDE COUNTY
REGIONAL CONSERVATION AUTHORITY
APN 423-240-027
0 40 80 120
SCALE: 1" = 40'
ENGINEERING COMPANY
NCM ENGINEERING CORP
4740 GREEN RIVER ROAD
SUITE 218
CORONA, CA 92880
PREPARED BY:
R.C.E. NO.
DATE
GILMAN SPRINGS RD
WIDENING
SHEET 2 OF 7

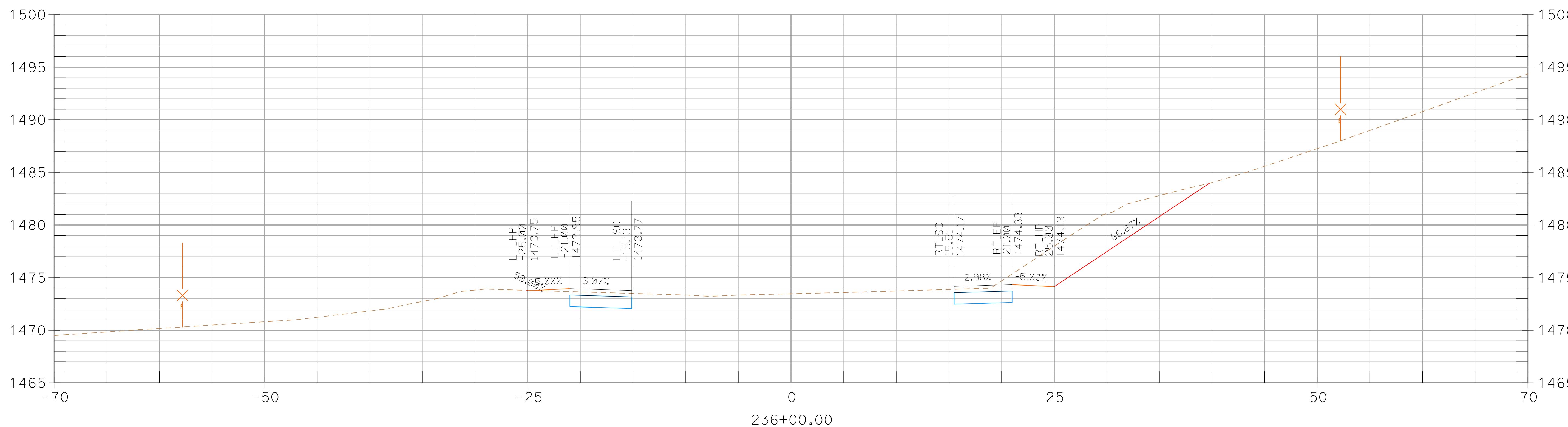


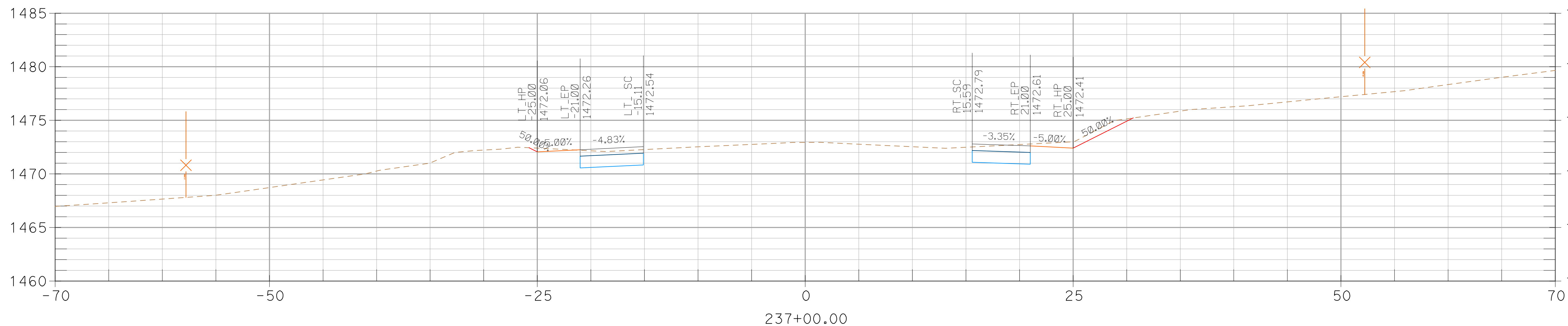
LT_HP -25.20 1473.98
LT_EP -21.20 1474.18
LT_SC -15.23 1474.37
RT_SC 16.70 1475.17
RT_EP 22.26 1475.27
RT_HP 26.26 1475.07
-3.15%
1.92%
-5.00%
50.00%
1490
1485
1480
1475
1470
1465
-70
-50
-25
0
25
50
70
235+00.00

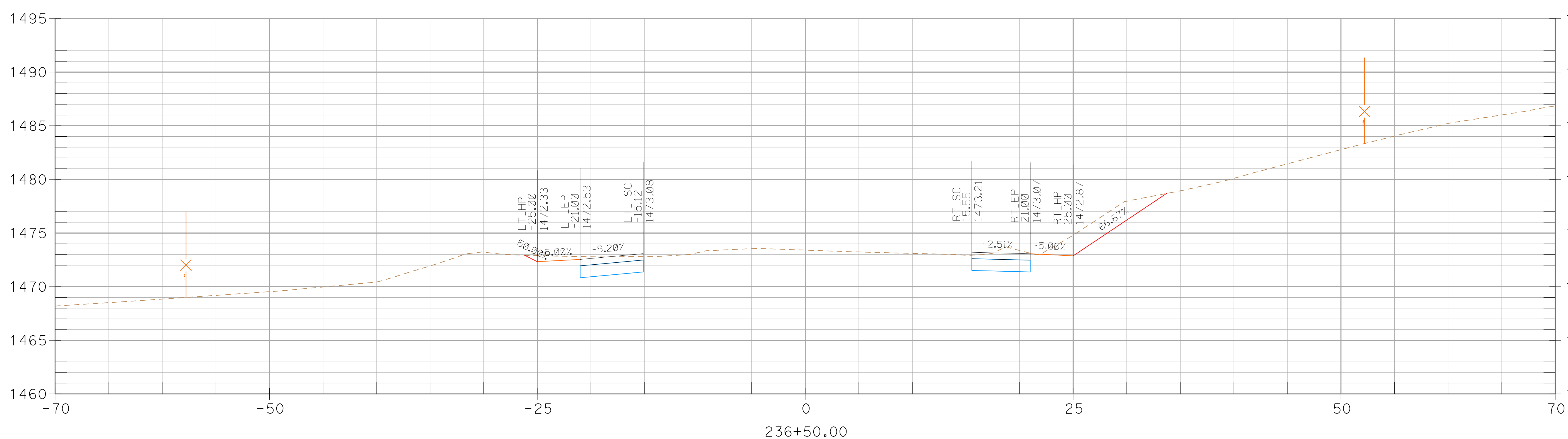


LT_HP -30.40 1474.22
LT_EP -26.40 1474.42
LT_SC -20.37 1474.59
RT_SC 17.48 1474.92
RT_EP 23.03 1474.72
RT_HP 27.03 1474.52
-50.00%
-5.00%
-2.75%
-3.53%
-5.00%
50.00%
1485
1480
1475
1470
1465
-70
-50
-25
0
25
50
70
234+50.00

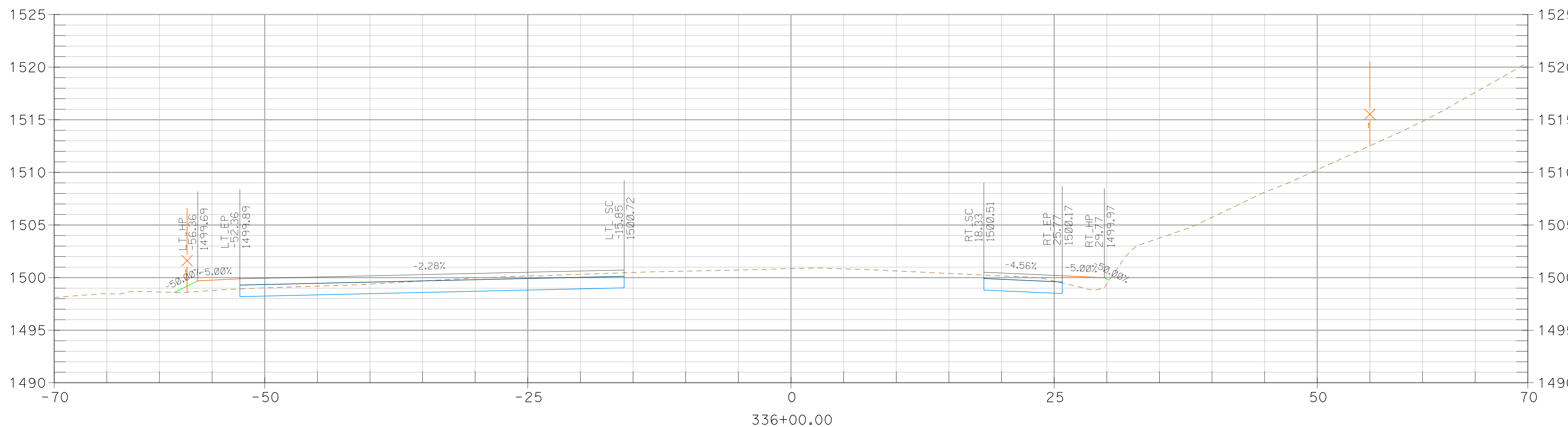

1495
1490
1485
1480
1475
1470
1465
LT_HP
-25.00
1473.62
LT_EP
-21.00
1473.82
LT_SC
-15.16
1474.03
RT_SC
15.69
1474.01
RT_EP
21.47
1473.77
RT_HP
25.47
1473.57
50.00%
-5.00%
-3.67%
-4.10%
-5.00%
66.67%
-70
-50
-25
0
25
50
70
235+50.00

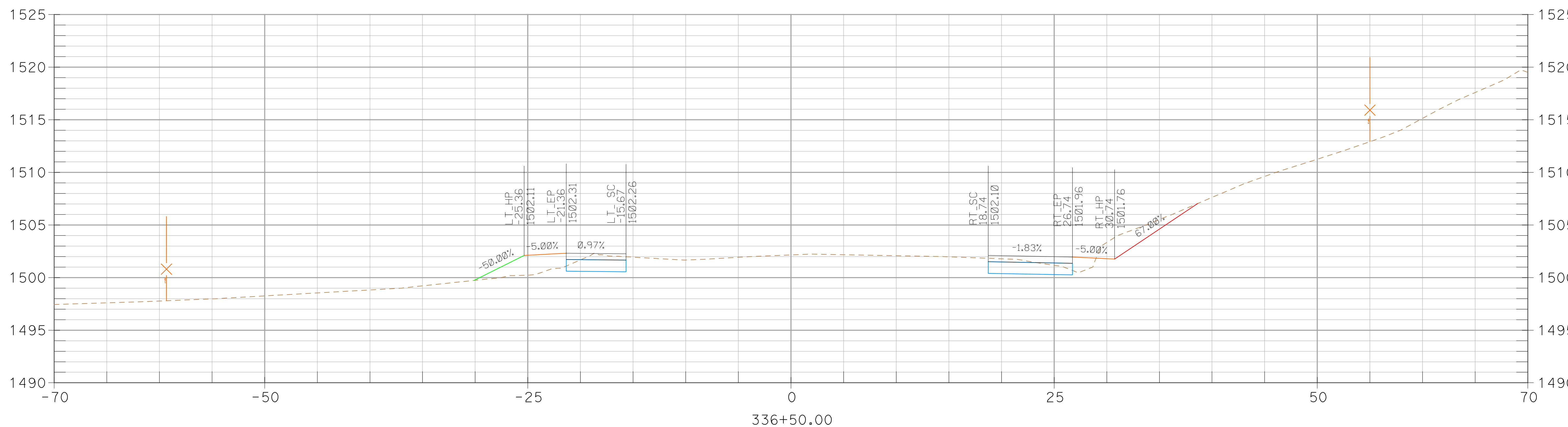

1500
1495
1490
1485
1480
1475
1470
1465
LT_HP -25.00 1473.75
LT_EP -21.00 1473.95
LT_SC -15.13 1473.77
RT_SC 15.51 1474.17
RT_EP 21.00 1474.33
RT_HP 25.00 1474.13
50.00%
5.00%
3.07%
2.98%
-5.00%
66.67%
-70
-50
-25
0
25
50
70
236+00.00

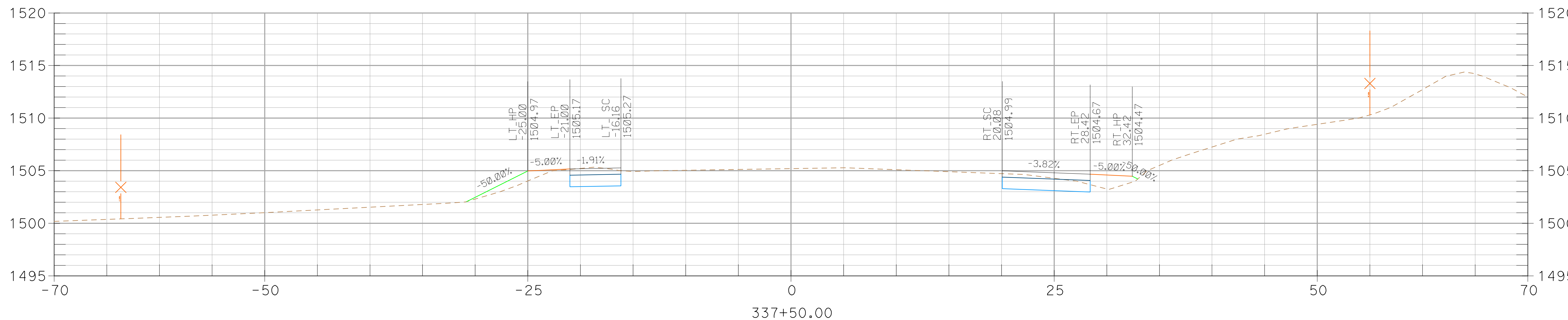

LT_HP
-25.00
1472.06
LT_EP
-21.00
1472.26
LT_SC
-15.11
1472.54
RT_SC
15.59
1472.79
RT_EP
21.00
1472.61
RT_HP
25.00
1472.41
50.00%
-5.00%
-4.83%
-3.35%
-5.00%
50.00%
1485
1480
1475
1470
1465
1460
-70
-50
-25
0
25
50
70
237+00.00

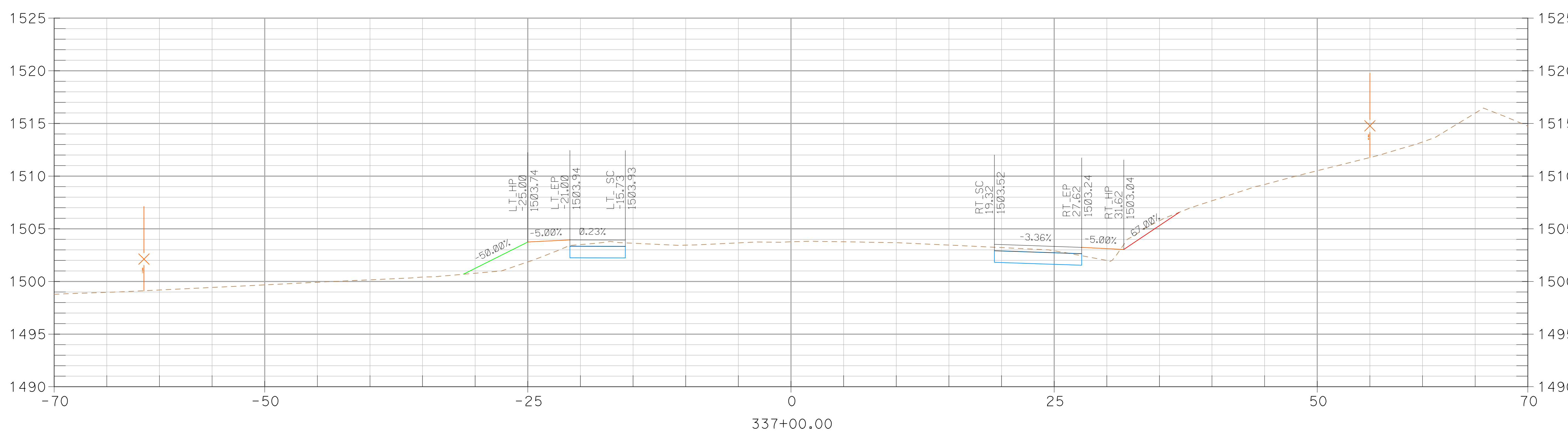

LT_HP
-25.00
1472.33
LT_EP
-21.00
1472.53
LT_SC
-15.12
1473.08
RT_SC
15.55
1473.21
RT_EP
21.00
1473.07
RT_HP
25.00
1472.87
50.00%
-5.00%
-9.20%
-2.51%
-5.00%
66.67%
1495
1490
1485
1480
1475
1470
1465
1460
-70
-50
-25
0
25
50
70
236+50.00


STATE OF CALIFORNIA
WILDLIFE CONSERVATION BOARD
APN 423-190-030
CENTRAL AVE
GILMAN SPRINGS ROAD LLC,
APN 423-180-009
GILMAN SPRINGS ROAD LLC,
APN 423-180-008
SEE MATCH LINE SHEET 4
314+50 "G" LINE
GILMAN SPRINGS RD
EXIST R = 1312.34'
SEE MATCH LINE BELOW
331+50 "G" LINE
(SLOPE)
R B VENTURES
APN 422-240-014
MICHAEL A. VAN RYN
DINA M. VAN RYN
APN 423-180-012
R B VENTURES
APN 423-190-001
GILMAN SPRINGS ROAD LLC,
APN 423-180-008
KNOCH RD
CURTIS ST
STATE OF CALIFORNIA
WILDLIFE CONSERVATION
BOARD
APN 423-180-005
STATE OF CALIFORNIA
WILDLIFE CONSERVATION BOARD
APN 423-180-006
SEE MATCH LINE ABOVE
331+50 "G" LINE
EXIST R = 1312.34'
EXIST R = 2854.34'
GILMAN SPRINGS RD
EXIST R = 1312.34'
SEE MATCH LINE SHEET 6
348+50 "G" LINE
(SLOPE)
(SLOPE)
JACK RABBIT TRAIL
(R/W)
R B VENTURES
APN 423-240-014
MICHAEL A. VAN RYN
DINA M. VAN RYN
APN 423-180-012
STA 336+00
BEG SLOPE TRANS
STA 336+50
END SLOPE TRANS
BEG 1.5:1 SLOPE
STA 337+00
END 1.5:1 SLOPE
BEG SLOPE TRANS
STA 337+50
END SLOPE TRANS
21036 QUAIL
RANCH LLC
GENUS, L.P.
APN 423-100-016
WESTERN RIVERSIDE COUNTY
REGIONAL CONSERVATION AUTHORITY
APN 423-180-002
COUNTY OF RIVERSIDE
APN 423-180-003
LEGEND
EXISTING R/W
EXISTING EASMENT (TYPE)
EP
HINGE LINE
GRADING CUT LIMIT
GRADING FILL LIMIT
SLOPE TRANSITION
PROPOSED 1.5 :1 CUT SLOPE
PROPOSED 1 :1 CUT SLOPE
SCALE: 1" = 40'
GILMAN SPRINGS RD
WIDENING
SHEET 5 OF 7

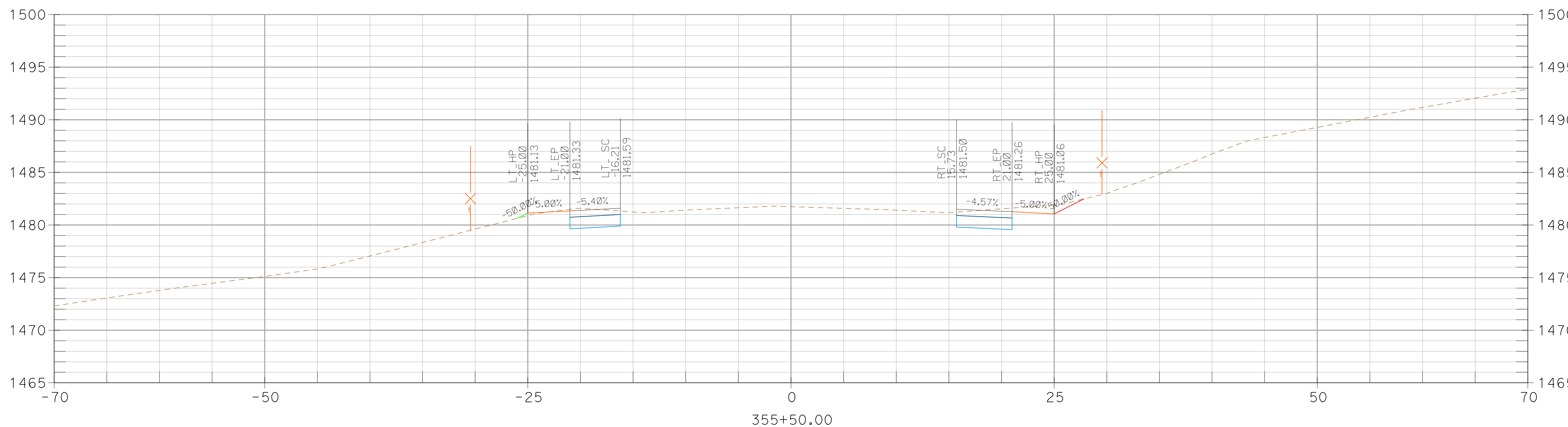

1525
1520
1515
1510
1505
1500
1495
1490
LT_HP
-56.36
1499.69
LT_EP
-52.36
1499.89
LT_SC
-15.85
1500.72
RT_SC
18.33
1500.51
RT_EP
25.77
1500.17
RT_HP
29.77
1499.97
-50.00%
-5.00%
-2.28%
-4.56%
-5.00%
-50.00%
-70
-50
-25
0
25
50
70
336+00.00


LT_HP
-25.36
1502.11
LT_EP
-21.36
1502.31
LT_SC
-15.67
1502.26
RT_SC
18.74
1502.10
RT_EP
26.74
1501.96
RT_HP
30.74
1501.76
-50.00%
-5.00%
0.97%
-1.83%
-5.00%
67.00%
1525
1520
1515
1510
1505
1500
1495
1490
-70
-50
-25
0
25
50
70
336+50.00

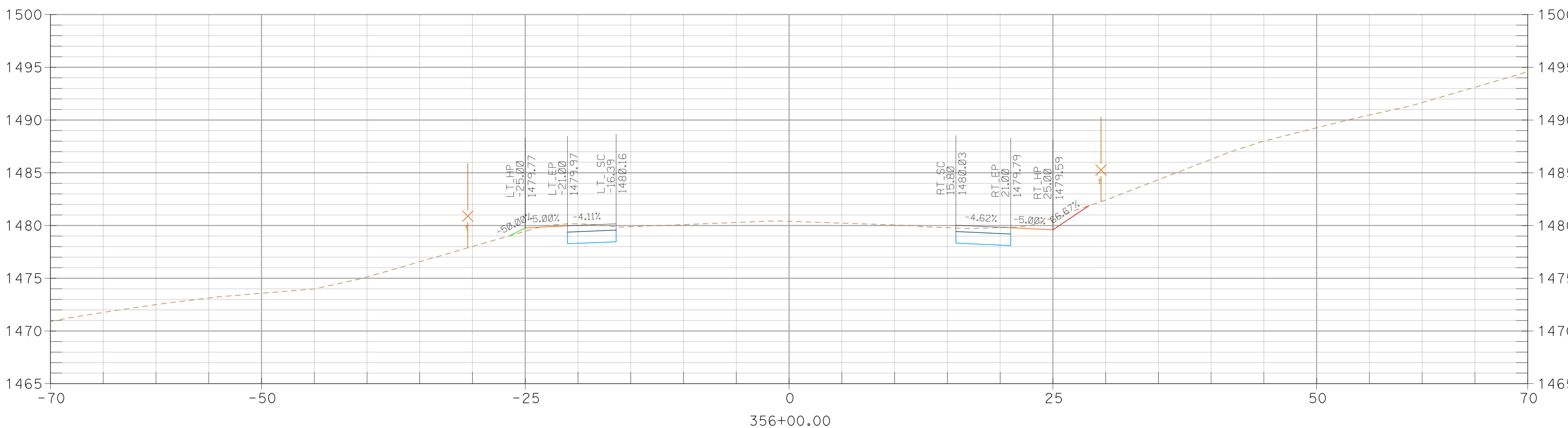

337+50.00
LT_HP -25.00 1504.97
LT_EP -21.00 1505.17
LT_SC -16.16 1505.27
RT_SC 20.08 1504.99
RT_EP 28.42 1504.67
RT_HP 32.42 1504.47
-50.00%
-5.00%
-1.91%
-3.82%
-5.00%
50.00%
1520
1515
1510
1505
1500
1495
-70
-50
-25
0
25
50
70

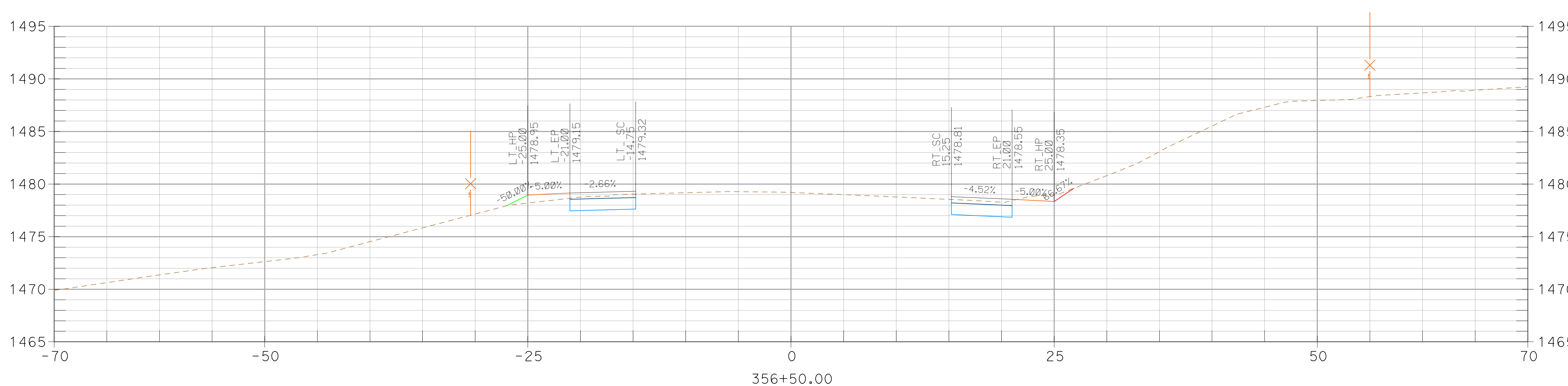

337+00.00
LT_HP -25.00 1503.74
LT_EP -21.00 1503.94
LT_SC -15.73 1503.93
RT_SC 19.32 1503.52
RT_EP 27.62 1503.24
RT_HP 31.62 1503.04
-50.00%
-5.00%
0.23%
-3.36%
-5.00%
67.00%
1525
1520
1515
1510
1505
1500
1495
1490
-70
-50
-25
0
25
50
70


STATE OF CALIFORNIA
WILDLIFE CONSERVATION BOARD
APN 423-180-006
MORENO GILMAN 650, LLC
APN 423-100-017
348+90.00
BEG RET WALL
351+60.00
END RET WALL
STATE OF CALIFORNIA
WILDLIFE CONSERVATION BOARD
APN 423-100-020
MORENO GILMAN 650, LLC
APN 423-100-019
EXIST R = 2854.34'
GILMAN SPRINGS RD
EXIST R = 1312.34'
(SLOPE)
(SLOPE)
(SLOPE)
SEE MATCH LINE SHEET 5
348+50 "G" LINE
21036 QUAIL RANCH LLC
GENUS, L.P.
APN 423-100-016
STA 355+50
BEG SLOPE TRANS
STA 356+00
END SLOPE TRANS
BEG 1.5:1 SLOPE
STA 361+00
END 1.5:1 SLOPE
BEG SLOPE TRANS
STA 361+50
END SLOPE TRANS
SEE MATCH LINE BELOW
366+50 "G" LINE
STATE OF CALIFORNIA
WILDLIFE CONSERVATION BOARD
APN 423-100-020
AHOME REAL ESTATE, LLC
APN 423-100-021
21036 QUAIL RANCH LLC
GENUS, L.P.
APN 423-100-015
STATE OF CALIFORNIA
WILDLIFE CONSERVATION BOARD
APN 423-080-015
MORENO GILMAN 650, LLC
APN 423-100-019
GILMAN SPRINGS RD
EXIST R = 1049.88'
AHOME REAL ESTATE, LLC
APN 423-080-016
SEE MATCH LINE ABOVE
366+50 "G" LINE
QUAIL RANCH
GOLF COURSE
21036 QUAIL RANCH LLC
GENUS, L.P.
APN 423-100-015
EXIST R = 2844.59'
SEE MATCH LINE SHEET 7
383+50 "G" LINE
21036 QUAIL RANCH LLC
GENUS, L.P.
APN 423-080-005
LEGEND
EXISTING R/W
EXISTING EASMENT (TYPE)
EP
HINGE LINE
GRADING CUT LIMIT
GRADING FILL LIMIT
SLOPE TRANSITION
PROPOSED 1.5 :1 CUT SLOPE
PROPOSED 1 :1 CUT SLOPE
0 40 80 120
SCALE: 1" = 40'
GILMAN SPRINGS RD
WIDENING
SHEET 6 OF 7

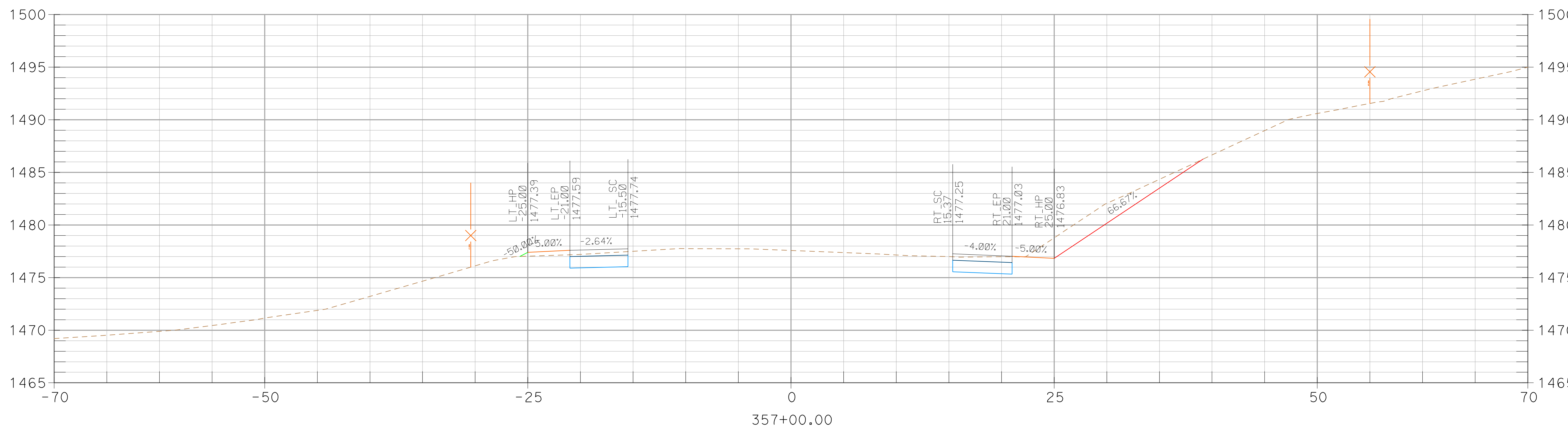


1500
1495
1490
1485
1480
1475
1470
1465
LT_HP -25.00 1481.13
LT_EP -21.00 1481.33
LT_SC -16.21 1481.59
RT_SC 15.73 1481.50
RT_EP 21.00 1481.26
RT_HP 25.00 1481.06
-50.00%
-5.00%
-5.40%
-4.57%
-5.00%
-50.00%
-70
-50
-25
0
25
50
70
355+50.00

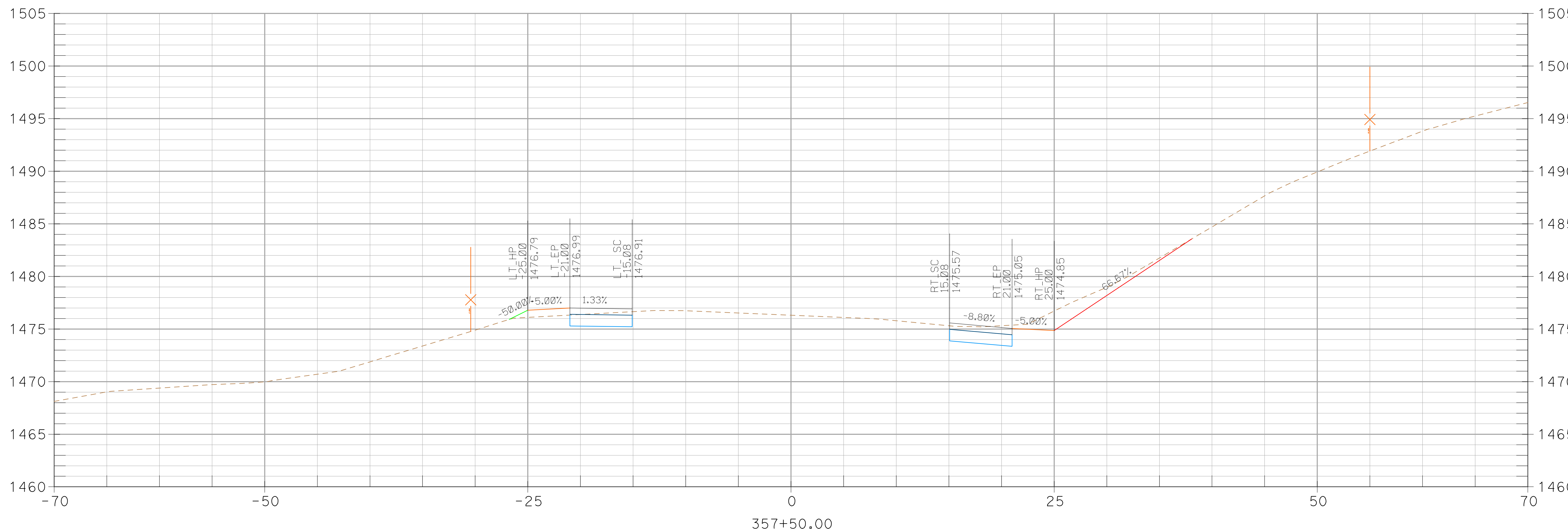

1500
1495
1490
1485
1480
1475
1470
1465
LT_HP -25.00 1479.77
LT_EP -21.00 1479.97
LT_SC -16.39 1480.16
RT_SC 15.80 1480.03
RT_EP 21.00 1479.79
RT_HP 25.00 1479.59
-50.00%
-5.00%
-4.11%
-4.62%
-5.00%
66.67%
-70
-50
-25
0
25
50
70
356+00.00



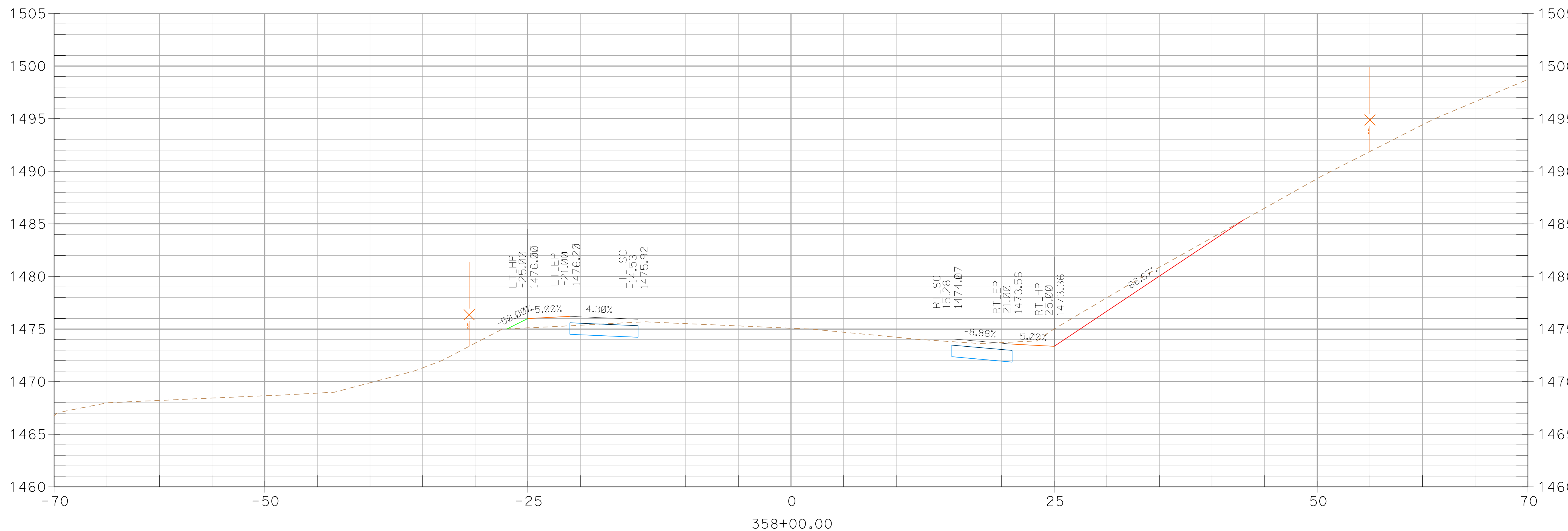
LT_HP
-25.00
1478.95
LT_EP
-21.00
1479.15
LT_SC
-14.75
1479.32
RT_SC
15.25
1478.81
RT_EP
21.00
1478.55
RT_HP
25.00
1478.35
-50.00%
-5.00%
-2.66%
-4.52%
-5.00%
66.67%
1495
1490
1485
1480
1475
1470
1465
-70
-50
-25
0
25
50
70
356+50.00

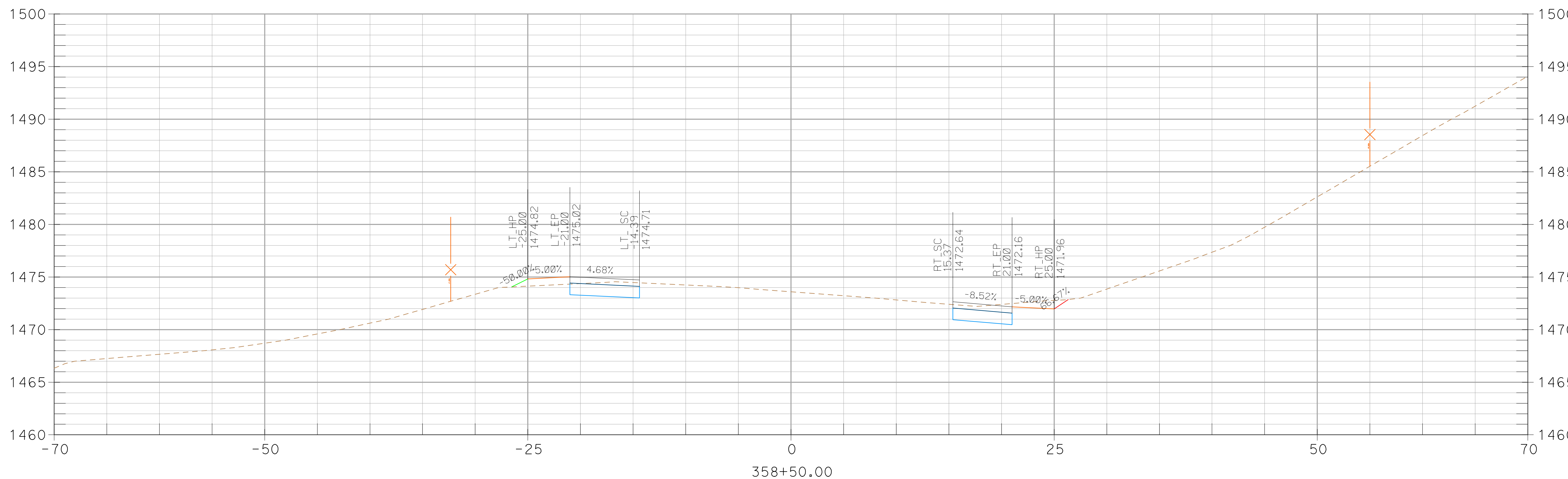


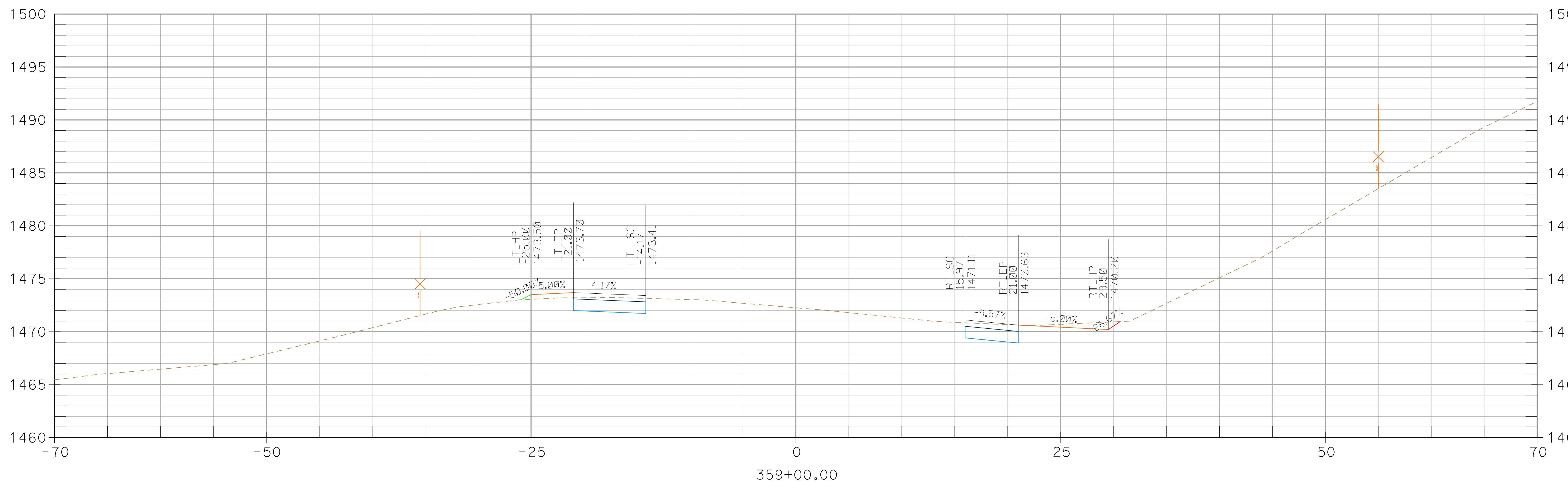
LT_HP
-25.00
1477.39
LT_EP
-21.00
1477.59
LT_SC
-15.50
1477.74
RT_SC
15.37
1477.25
RT_EP
21.00
1477.03
RT_HP
25.00
1476.83
-50.00%
-5.00%
-2.64%
-4.00%
-5.00%
66.67%
1500
1495
1490
1485
1480
1475
1470
1465
-70
-50
-25
0
25
50
70
357+00.00



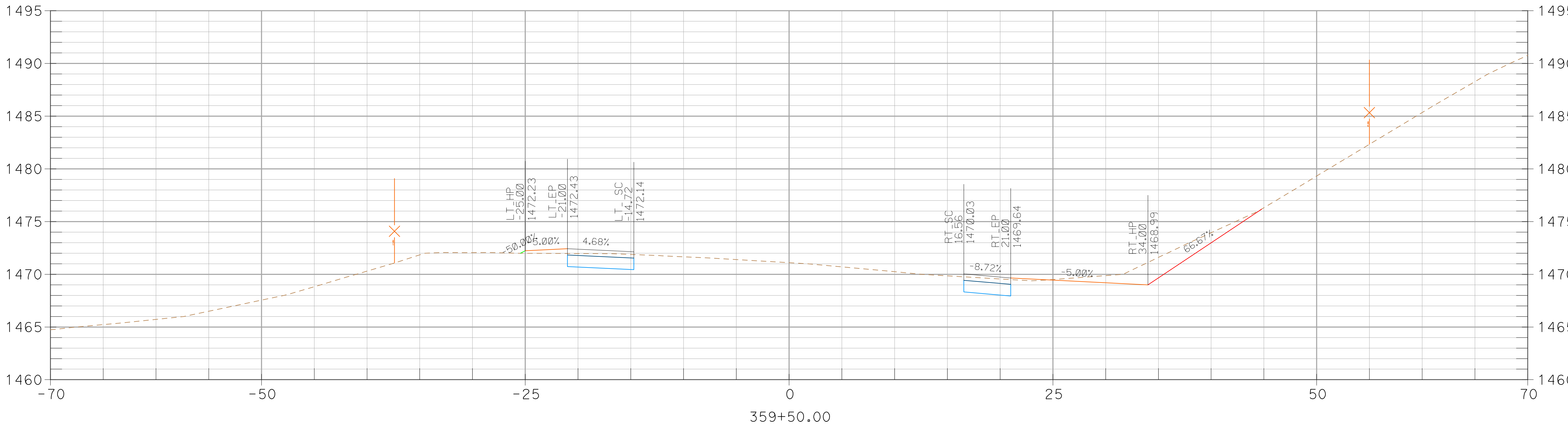
1505
1500
1495
1490
1485
1480
1475
1470
1465
1460
-70
-50
-25
0
25
50
70
357+50.00
LT_HP
-25.00
1476.79
LT_EP
-21.00
1476.99
LT_SC
-15.08
1476.91
-50.00%
-5.00%
1.33%
RT_SC
15.08
1475.57
RT_EP
21.00
1475.05
RT_HP
25.00
1474.85
-8.80%
-5.00%
66.67%

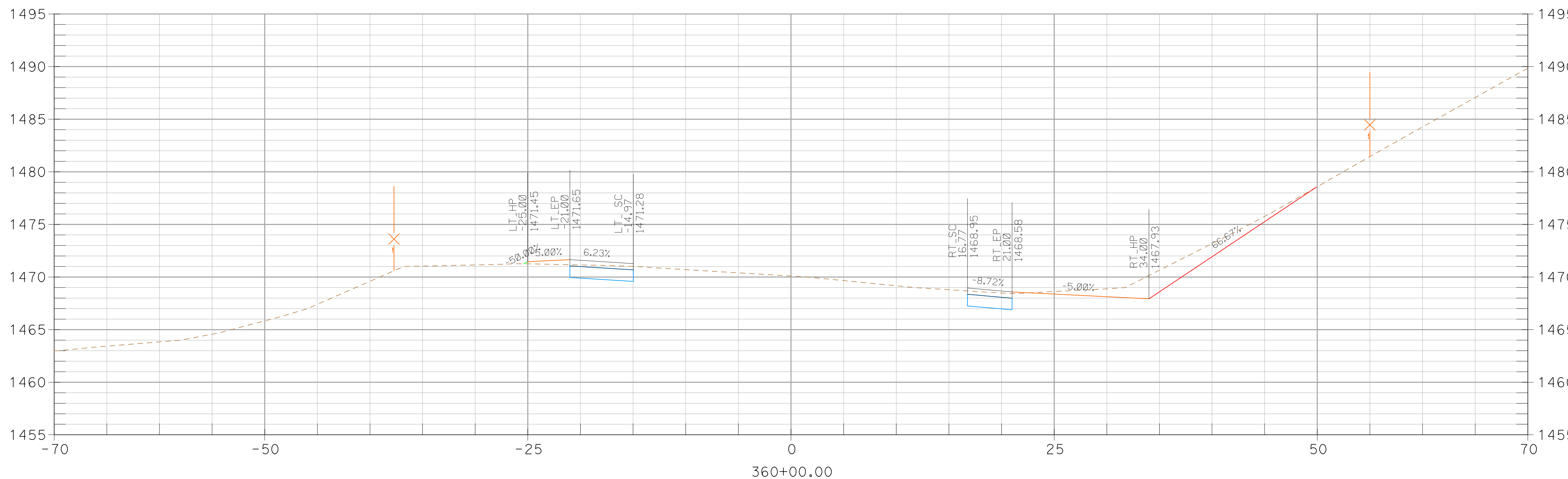

1505
1500
1495
1490
1485
1480
1475
1470
1465
1460
LT_HP
-25.00
1476.00
LT_EP
-21.00
1476.20
LT_SC
-14.53
1475.92
-50.00%
-5.00%
4.30%
RT_SC
15.28
1474.07
RT_EP
21.00
1473.56
RT_HP
25.00
1473.36
-8.88%
-5.00%
66.67%
-70
-50
-25
0
25
50
70
358+00.00

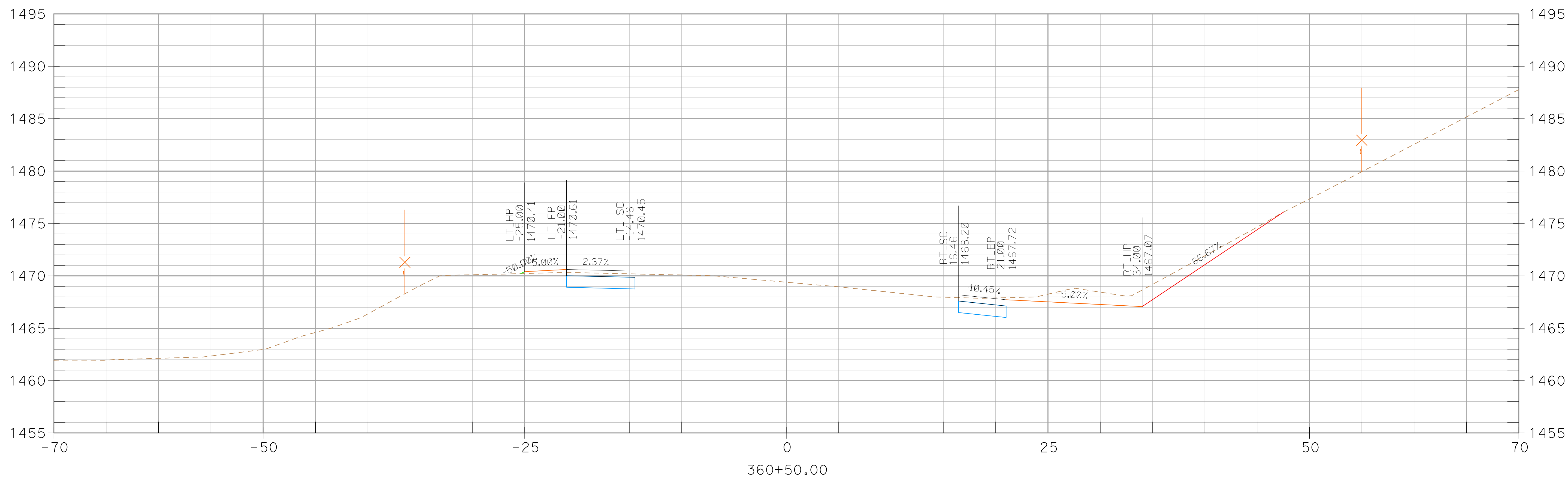

LT_HP
-25.00
1474.82
LT_EP
-21.00
1475.02
LT_SC
-14.39
1474.71
RT_SC
15.37
1472.64
RT_EP
21.00
1472.16
RT_HP
25.00
1471.96
-50.00%
-5.00%
4.68%
-8.52%
-5.00%
66.67%
1500
1495
1490
1485
1480
1475
1470
1465
1460
-70
-50
-25
0
25
50
70
358+50.00

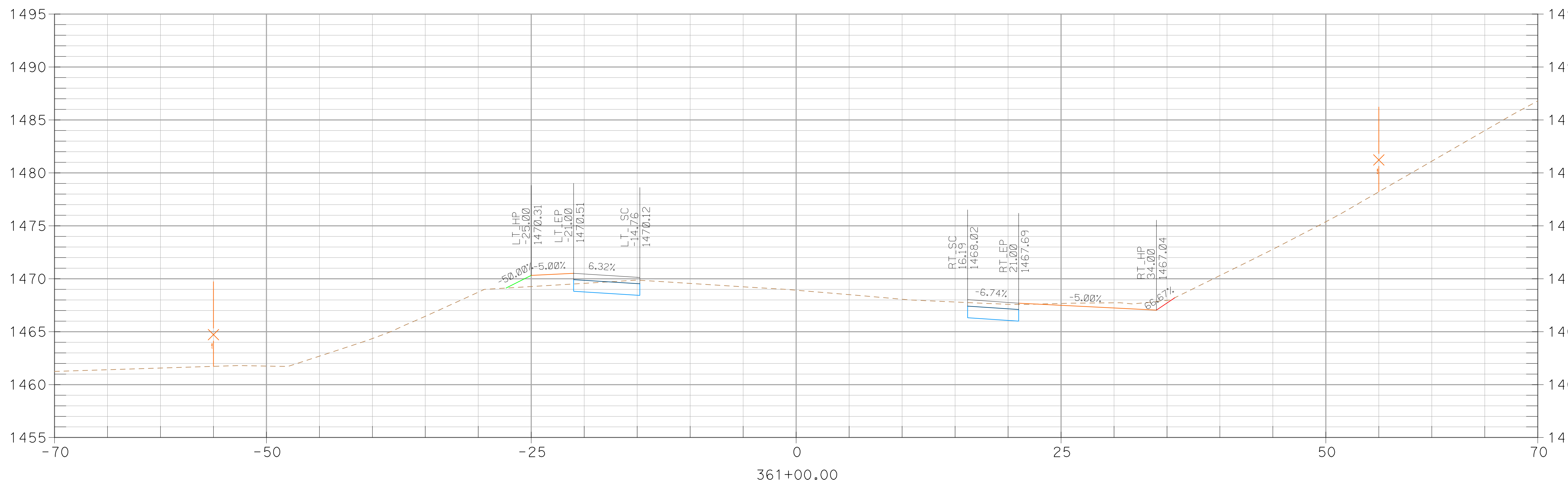


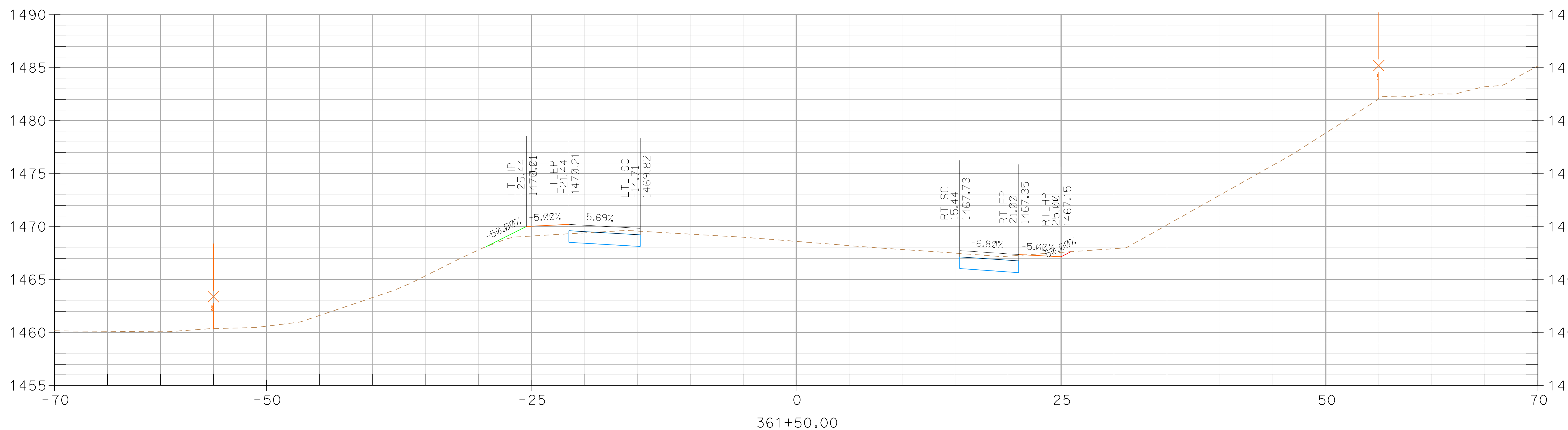
1500
1495
1490
1485
1480
1475
1470
1465
1460
-70
-50
-25
0
25
50
70
359+00.00
LT_HP
-25.00
1473.50
LT_EP
-21.00
1473.70
LT_SC
-14.17
1473.41
-50.00%
-5.00%
4.17%
RT_SC
15.97
1471.11
RT_EP
21.00
1470.63
RT_HP
29.50
1470.20
-9.57%
-5.00%
66.67%


1495
1490
1485
1480
1475
1470
1465
1460
LT_HP
-25.00
1472.23
LT_EP
-21.00
1472.43
LT_SC
-14.72
1472.14
-50.00%
5.00%
4.68%
RT_SC
16.56
1470.03
RT_EP
21.00
1469.64
-8.72%
-5.00%
RT_HP
34.00
1468.99
66.67%
-70
-50
-25
0
25
50
70
359+50.00

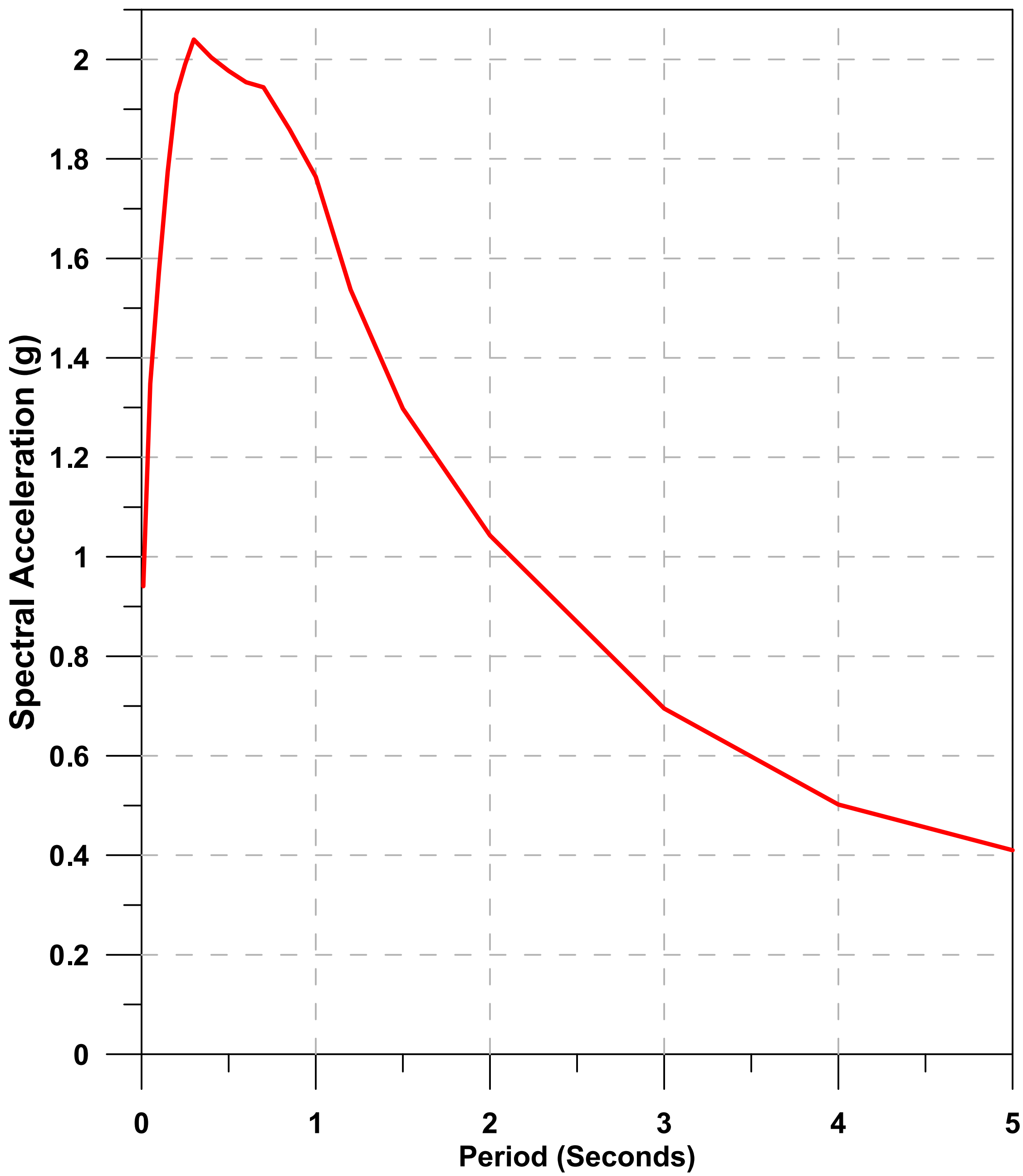

1495
1490
1485
1480
1475
1470
1465
1460
1455
-70
-50
-25
0
25
50
70
360+00.00
LT_HP
-25.00
1471.45
LT_EP
-21.00
1471.65
LT_SC
-14.97
1471.28
-50.00%
-5.00%
6.23%
RT_SC
16.77
1468.95
RT_EP
21.00
1468.58
-8.72%
-5.00%
RT_HP
34.00
1467.93
66.67%


LT_HP
-25.00
1470.41
LT_EP
-21.00
1470.61
LT_SC
-14.46
1470.45
-50.00%
-5.00%
2.37%
RT_SC
16.46
1468.20
RT_EP
21.00
1467.72
-10.45%
-5.00%
RT_HP
34.00
1467.07
66.67%
1495
1490
1485
1480
1475
1470
1465
1460
1455
-70
-50
-25
0
25
50
70
360+50.00


1495
1490
1485
1480
1475
1470
1465
1460
1455
LT_HP
-25.00
1470.31
LT_EP
-21.00
1470.51
LT_SC
-14.76
1470.12
-50.00%
-5.00%
6.32%
RT_SC
16.19
1468.02
RT_EP
21.00
1467.69
RT_HP
34.00
1467.04
-6.74%
-5.00%
66.67%
-70
-50
-25
0
25
50
70
361+00.00


LT_HP
-25.44
1470.01
LT_EP
-21.44
1470.21
LT_SC
-14.71
1469.82
-50.00%
-5.00%
5.69%
RT_SC
15.44
1467.73
RT_EP
21.00
1467.35
RT_HP
25.00
1467.15
-6.80%
-5.00%
50.00%
1490
1485
1480
1475
1470
1465
1460
1455
-70
-50
-25
0
25
50
70
361+50.00

APPENDIX E - CALCULATIONS



Notes:
1. Location: Latitude = 33.889647° N, Longitude = 117.071229° W.
2. Vs30 = 300 m/s.
3. Damping = 5%.
4. The ARS is developed using Caltrans ARS Online V.2.3.09.
5. The ARS shown is an envelope of deterministic and probabilistic spectra.

PRELIMINARY DESIGN HORIZONTAL ACCELERATION RESPONSE SPECTRUM - Gilman Springs Road Improvements

SITE DATA (ARS Online Version 2.3.09)

Shear Wave Velocity, V_{S30}:	300 m/s
Latitude:	33.889647
Longitude:	-117.071229
Depth to Vs = 1.0 km/s:	N/A
Depth to Vs = 2.5 km/s:	N/A

DETERMINISTIC

San Jacinto (San Jacinto Valley)

Fault ID:	356
Maximum Magnitude (MMax):	7.7
Fault Type:	SS
Fault Dip:	90 Deg
Dip Direction:	V
Bottom of Rupture Plane:	16.00 km
Top of Rupture Plane(Ztor):	0.00 km
Rrup	0.05 km
Rjb:	0.05 km
Rx:	0.05 km
Fnorm:	0
Frev:	0

Period	SA(Base Spectrum)	Basin Factor	Near Fault Factor(Applied)	SA(Final Spectrum)
0.01	0.540	1.000	1.000	0.540
0.05	0.639	1.000	1.000	0.639
0.1	0.793	1.000	1.000	0.793
0.15	0.923	1.000	1.000	0.923
0.2	1.025	1.000	1.000	1.025
0.25	1.081	1.000	1.000	1.081
0.3	1.108	1.000	1.000	1.108
0.4	1.125	1.000	1.000	1.125
0.5	1.133	1.000	1.000	1.133
0.6	1.089	1.000	1.040	1.132

0.7	1.049	1.000	1.080	1.133
0.85	0.977	1.000	1.140	1.114
1	0.905	1.000	1.200	1.086
1.2	0.813	1.000	1.200	0.975
1.5	0.700	1.000	1.200	0.840
2	0.549	1.000	1.200	0.659
3	0.360	1.000	1.200	0.433
4	0.261	1.000	1.200	0.313
5	0.203	1.000	1.200	0.243

San Jacinto (Anza)

Fault ID:	362
Maximum Magnitude (MMax):	7.7
Fault Type:	SS
Fault Dip:	90 Deg
Dip Direction:	V
Bottom of Rupture Plane:	17.00 km
Top of Rupture Plane(Ztor):	0.00 km
Rrup	3.20 km
Rjb:	3.20 km
Rx:	3.20 km
Fnorm:	0
Frev:	0

Period	SA(Base Spectrum)	Basin Factor	Near Fault Factor(Applied)	SA(Final Spectrum)
0.01	0.468	1.000	1.000	0.468
0.05	0.558	1.000	1.000	0.558
0.1	0.715	1.000	1.000	0.715
0.15	0.836	1.000	1.000	0.836
0.2	0.920	1.000	1.000	0.920
0.25	0.958	1.000	1.000	0.958
0.3	0.971	1.000	1.000	0.971
0.4	0.965	1.000	1.000	0.965
0.5	0.956	1.000	1.000	0.956
0.6	0.906	1.000	1.040	0.942
0.7	0.863	1.000	1.080	0.932

0.85	0.797	1.000	1.140	0.908
1	0.735	1.000	1.200	0.882
1.2	0.658	1.000	1.200	0.789
1.5	0.565	1.000	1.200	0.678
2	0.442	1.000	1.200	0.530
3	0.289	1.000	1.200	0.347
4	0.209	1.000	1.200	0.251
5	0.164	1.000	1.200	0.196

San Jacinto (San Bernardino Valley section)

Fault ID:	310
Maximum Magnitude (MMax):	7.7
Fault Type:	SS
Fault Dip:	90 Deg
Dip Direction:	V
Bottom of Rupture Plane:	15.00 km
Top of Rupture Plane(Ztor):	0.00 km
Rrup	12.96 km
Rjb:	12.96 km
Rx:	1.15 km
Fnorm:	0
Frev:	0

Period	SA(Base Spectrum)	Basin Factor	Near Fault Factor(Applied)	SA(Final Spectrum)
0.01	0.293	1.000	1.000	0.293
0.05	0.354	1.000	1.000	0.354
0.1	0.496	1.000	1.000	0.496
0.15	0.595	1.000	1.000	0.595
0.2	0.636	1.000	1.000	0.636
0.25	0.640	1.000	1.000	0.640
0.3	0.632	1.000	1.000	0.632
0.4	0.595	1.000	1.000	0.595
0.5	0.565	1.000	1.000	0.565
0.6	0.521	1.000	1.040	0.542
0.7	0.486	1.000	1.080	0.525
0.85	0.439	1.000	1.140	0.501

1	0.400	1.000	1.200	0.480
1.2	0.354	1.000	1.200	0.425
1.5	0.300	1.000	1.200	0.360
2	0.231	1.000	1.200	0.277
3	0.149	1.000	1.200	0.179
4	0.108	1.000	1.200	0.129
5	0.084	1.000	1.200	0.101

PROBABILISTIC

Probabilistic Model
USGS Seismic Hazard Map(2008) 975 Year Return Period

Period	SA(Base Spectrum)	Basin Factor	Near Fault Factor(Applied)	SA(Final Spectrum)
0.01	0.941	1.000	1.000	0.941
0.05	1.348	1.000	1.000	1.348
0.1	1.574	1.000	1.000	1.574
0.15	1.774	1.000	1.000	1.774
0.2	1.930	1.000	1.000	1.930
0.25	1.990	1.000	1.000	1.990
0.3	2.040	1.000	1.000	2.040
0.4	2.004	1.000	1.000	2.004
0.5	1.977	1.000	1.000	1.977
0.6	1.879	1.000	1.040	1.954
0.7	1.800	1.000	1.080	1.944
0.85	1.631	1.000	1.140	1.859
1	1.470	1.000	1.200	1.764
1.2	1.281	1.000	1.200	1.537
1.5	1.081	1.000	1.200	1.298
2	0.870	1.000	1.200	1.043
3	0.579	1.000	1.200	0.695
4	0.418	1.000	1.200	0.502
5	0.342	1.000	1.200	0.410

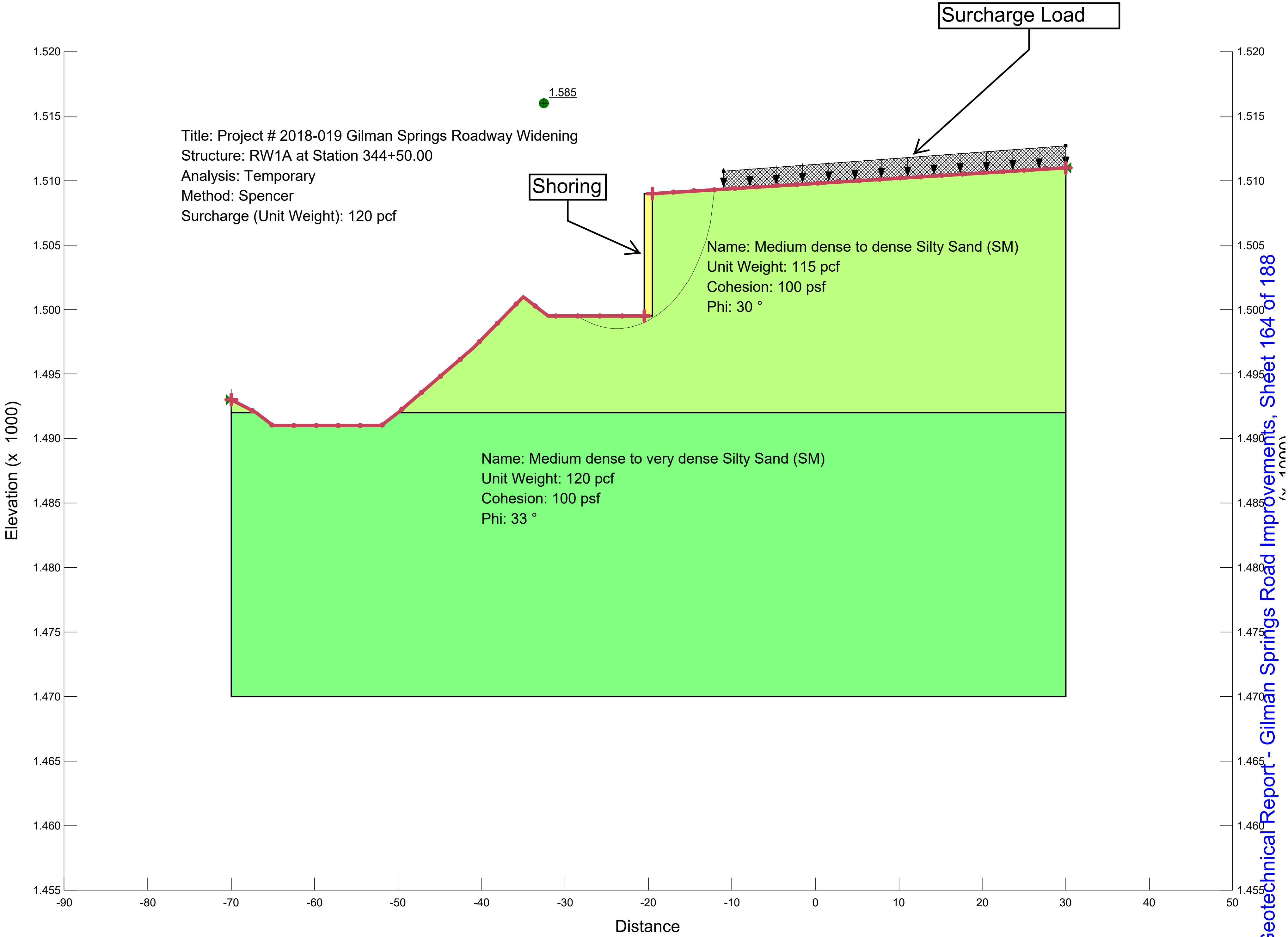
MINIMUM DETERMINISTIC SPECTRUM

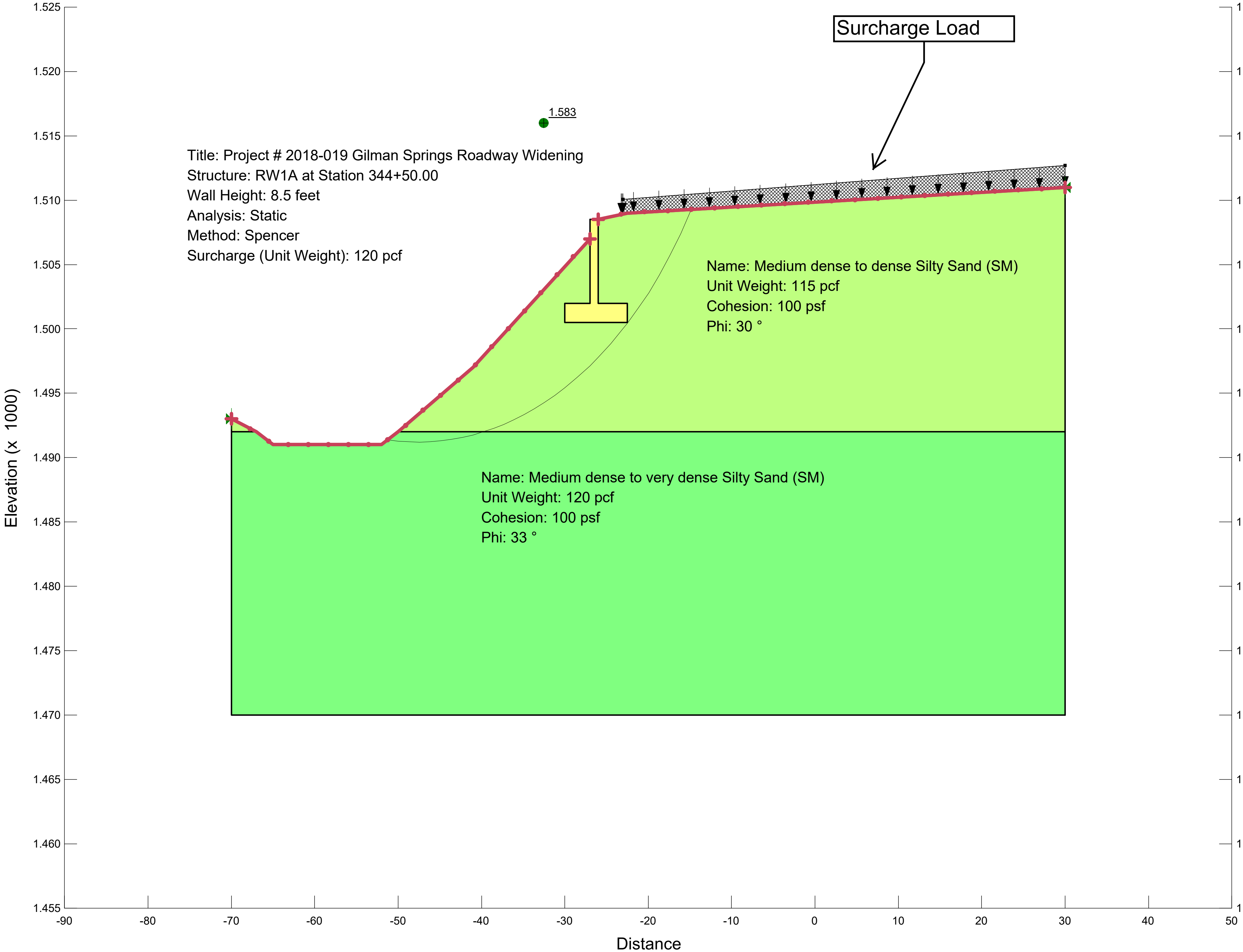
Period	SA
0.01	0.227
0.05	0.279
0.1	0.409
0.15	0.491
0.2	0.514
0.25	0.504
0.3	0.487
0.4	0.442
0.5	0.392
0.6	0.340
0.7	0.301
0.85	0.254
1	0.218
1.2	0.181
1.5	0.142
2	0.099
3	0.058
4	0.039
5	0.029

Envelope Data

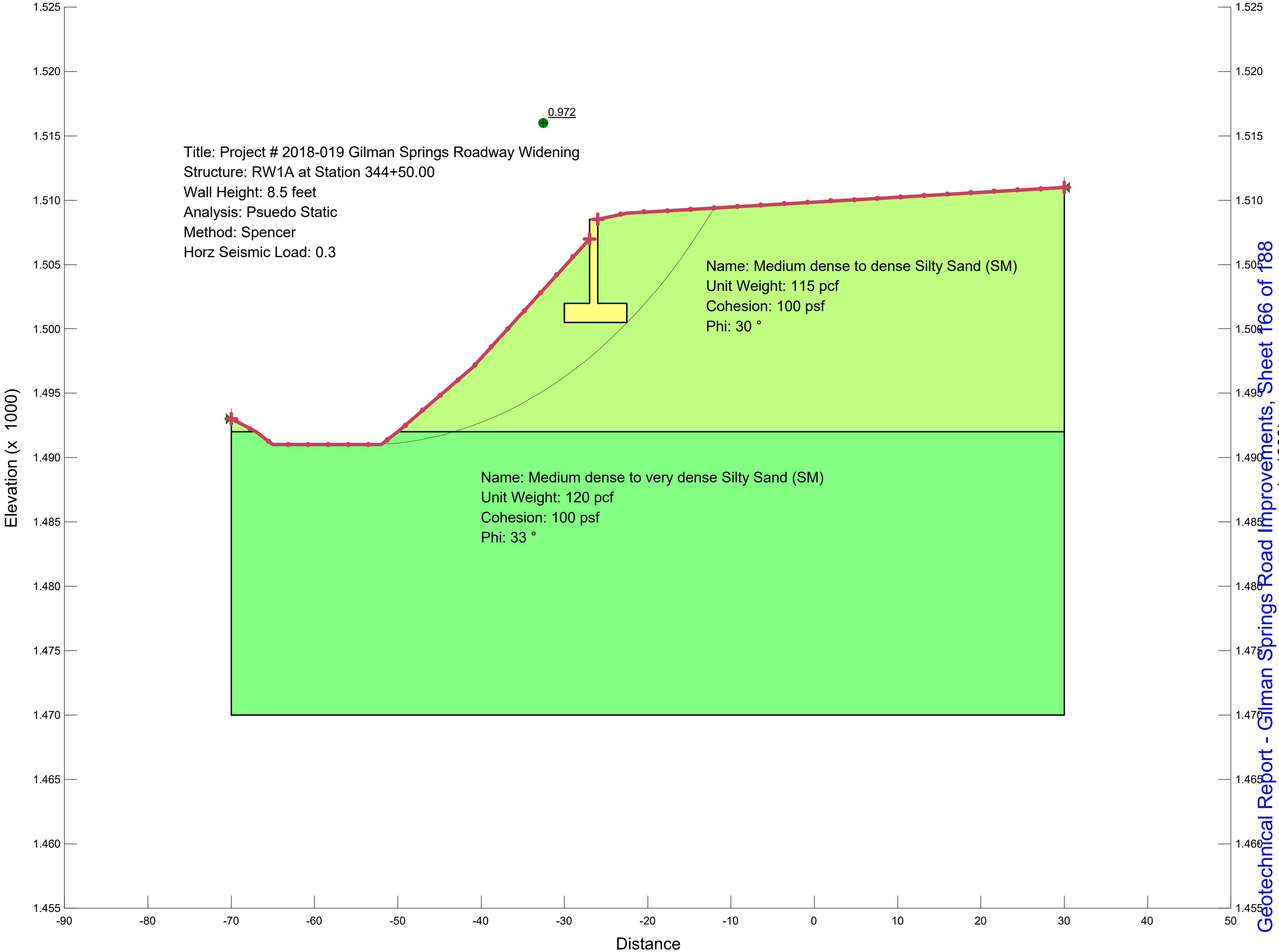
Period	SA
0.01	0.941
0.05	1.348
0.1	1.574
0.15	1.774
0.2	1.930
0.25	1.990
0.3	2.040
0.4	2.004
0.5	1.977
0.6	1.954
0.7	1.944

0.85	1.859
1	1.764
1.2	1.537
1.5	1.298
2	1.043
3	0.695
4	0.502
5	0.410

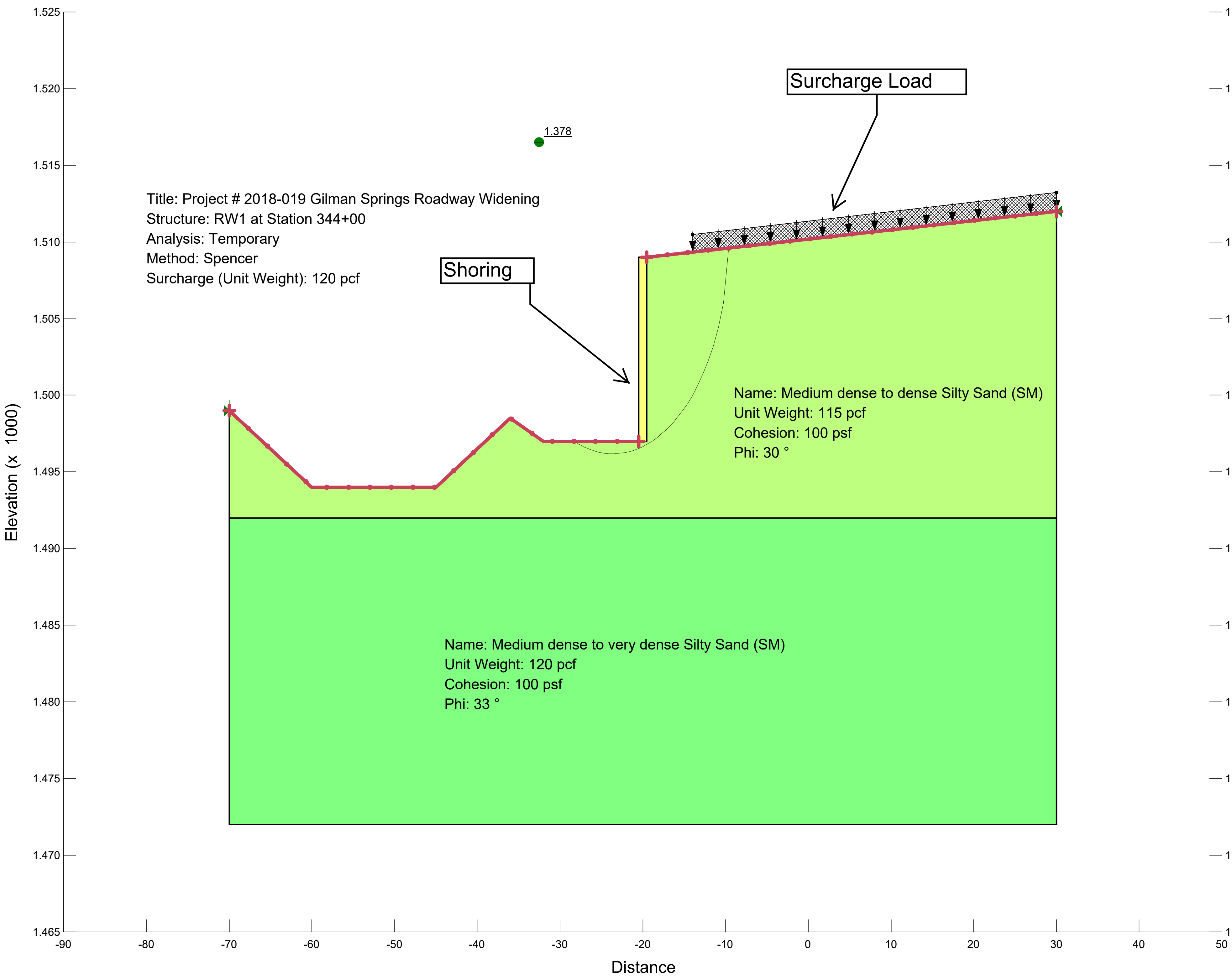

Surcharge Load
1.585
Title: Project # 2018-019 Gilman Springs Roadway Widening
Structure: RW1A at Station 344+50.00
Analysis: Temporary
Method: Spencer
Surcharge (Unit Weight): 120 pcf
Shoring
Name: Medium dense to dense Silty Sand (SM)
Unit Weight: 115 pcf
Cohesion: 100 psf
Phi: 30 °
Name: Medium dense to very dense Silty Sand (SM)
Unit Weight: 120 pcf
Cohesion: 100 psf
Phi: 33 °
Elevation (x 1000)
Distance
1.520
1.515
1.510
1.505
1.500
1.495
1.490
1.485
1.480
1.475
1.470
1.465
1.460
1.455
-90
-80
-70
-60
-50
-40
-30
-20
-10
0
10
20
30
40
50

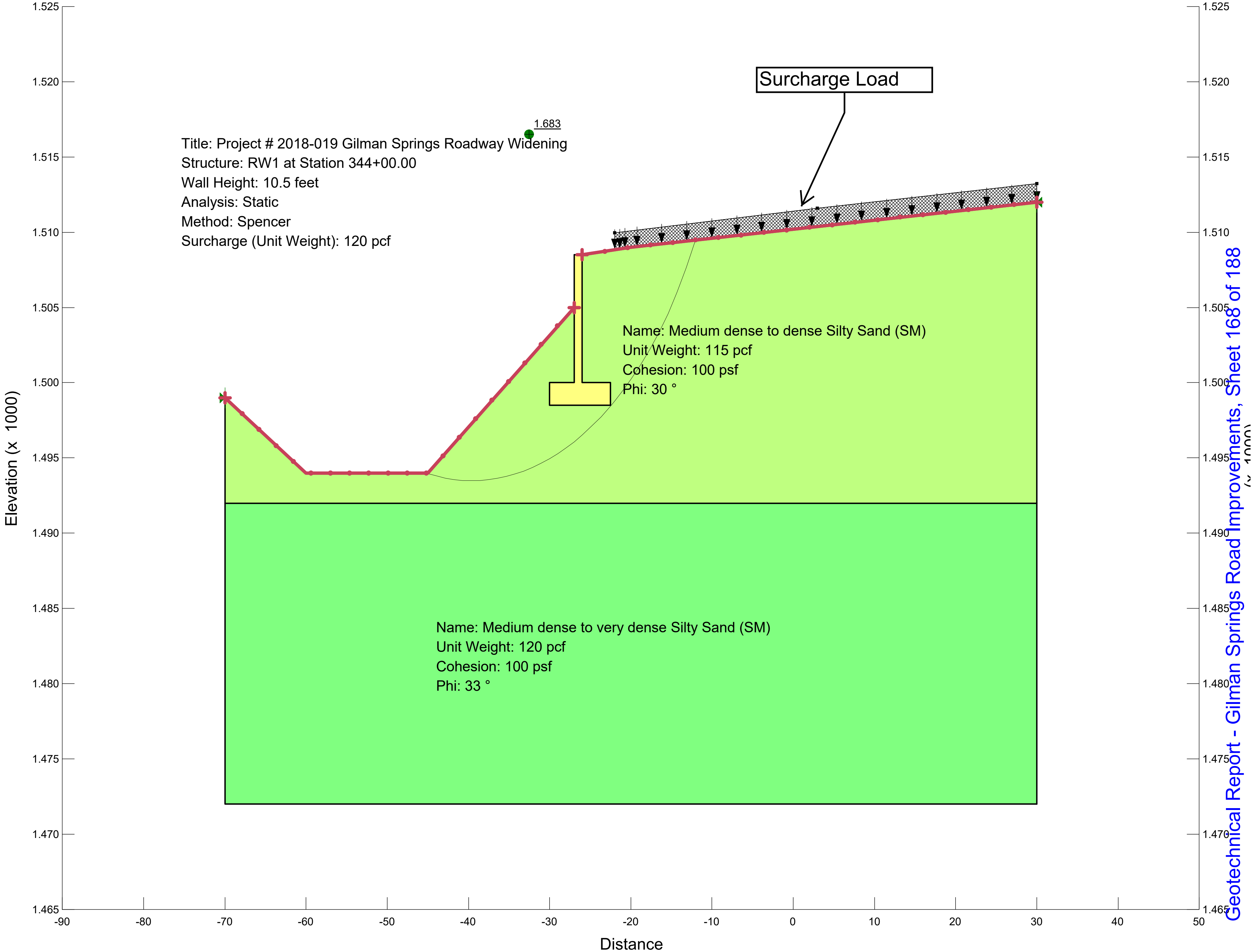


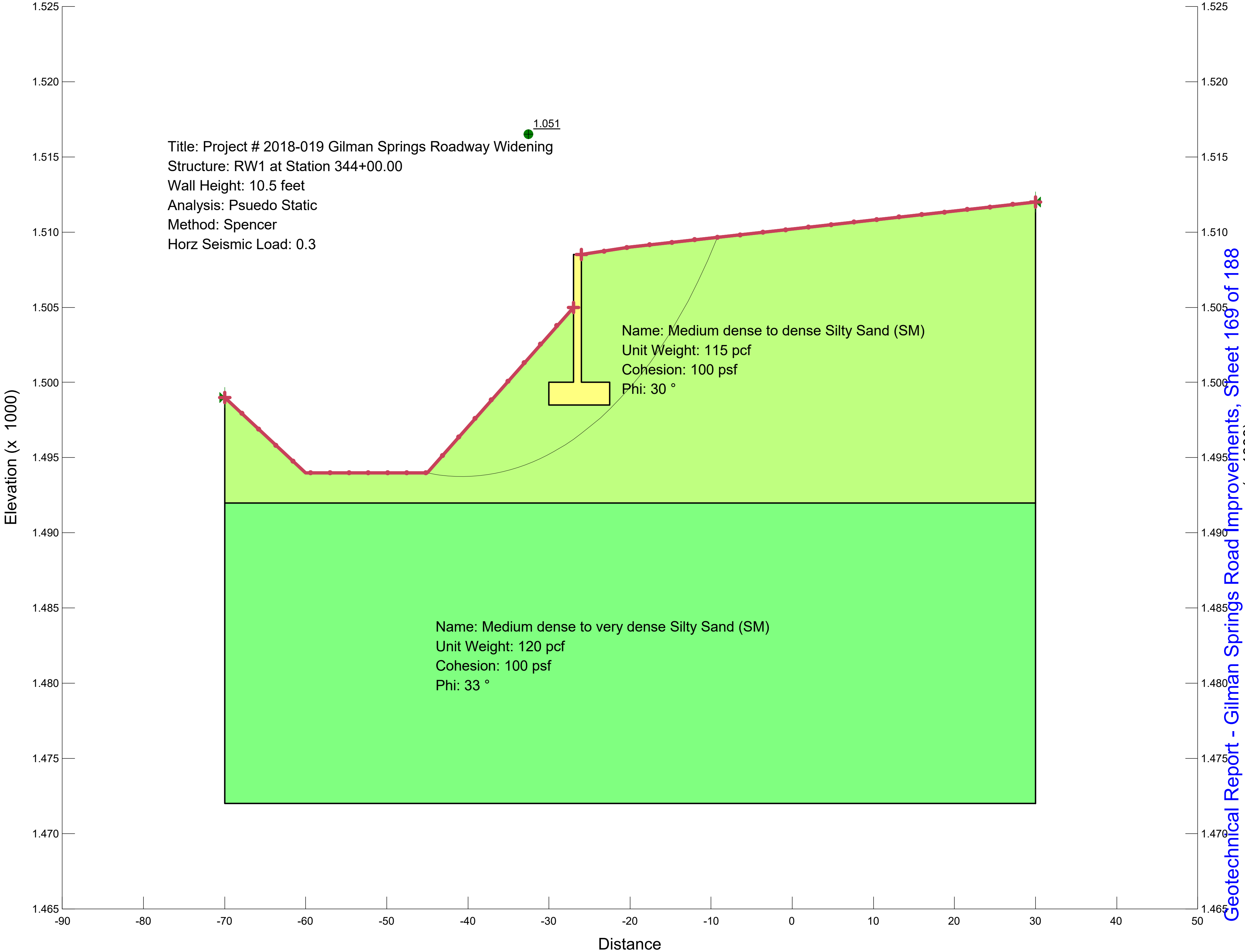
Surcharge Load
1.583
Title: Project # 2018-019 Gilman Springs Roadway Widening
Structure: RW1A at Station 344+50.00
Wall Height: 8.5 feet
Analysis: Static
Method: Spencer
Surcharge (Unit Weight): 120 pcf
Name: Medium dense to dense Silty Sand (SM)
Unit Weight: 115 pcf
Cohesion: 100 psf
Phi: 30 °
Name: Medium dense to very dense Silty Sand (SM)
Unit Weight: 120 pcf
Cohesion: 100 psf
Phi: 33 °
Elevation (x 1000)
Distance
1.525
1.520
1.515
1.510
1.505
1.500
1.495
1.490
1.485
1.480
1.475
1.470
1.465
1.460
1.455
-90
-80
-70
-60
-50
-40
-30
-20
-10
0
10
20
30
40
50

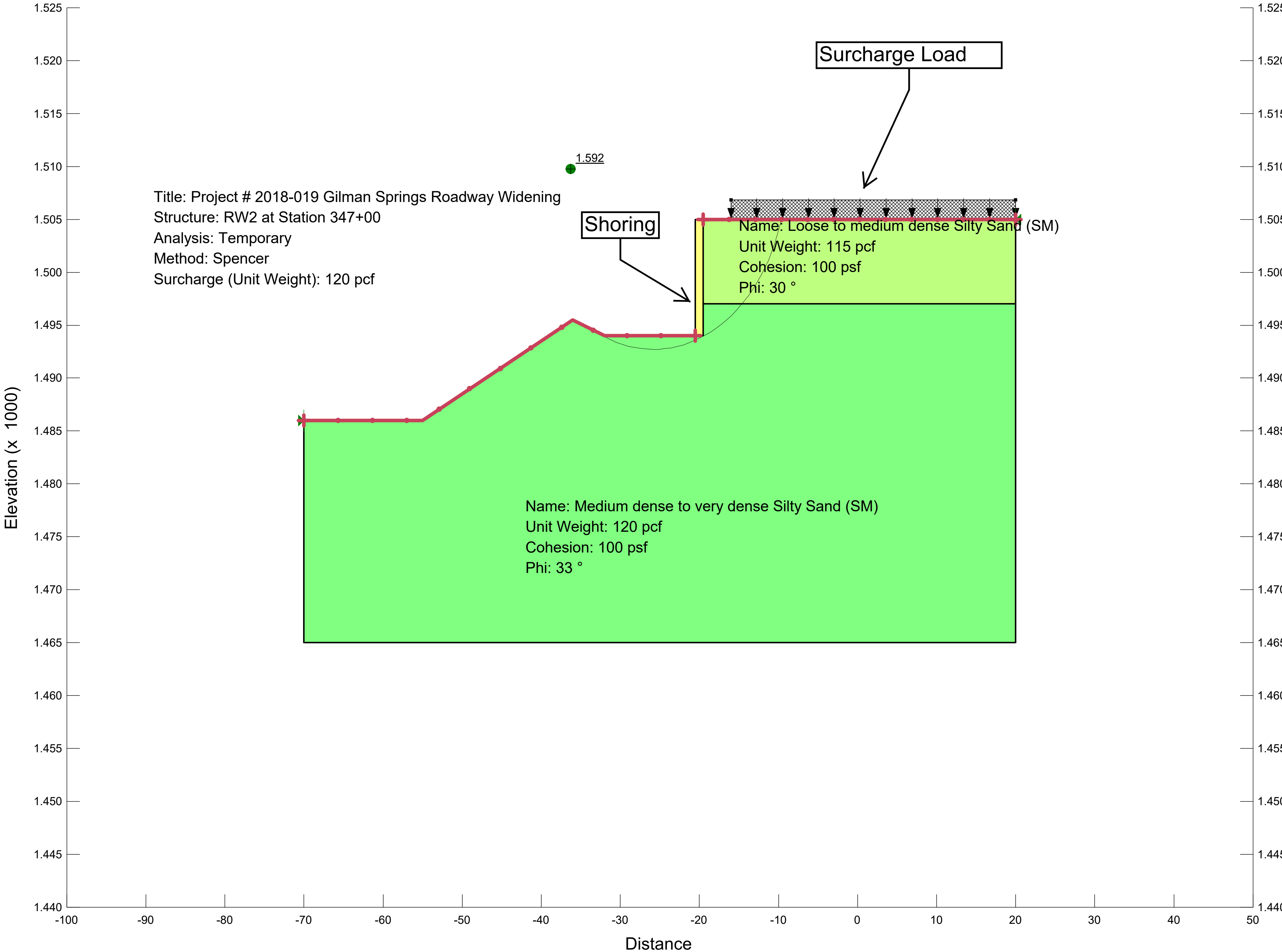


0.972
Title: Project # 2018-019 Gilman Springs Roadway Widening
Structure: RW1A at Station 344+50.00
Wall Height: 8.5 feet
Analysis: Psuedo Static
Method: Spencer
Horz Seismic Load: 0.3
Name: Medium dense to dense Silty Sand (SM)
Unit Weight: 115 pcf
Cohesion: 100 psf
Phi: 30 °
Name: Medium dense to very dense Silty Sand (SM)
Unit Weight: 120 pcf
Cohesion: 100 psf
Phi: 33 °
Elevation (x 1000)
Distance
1.525
1.520
1.515
1.510
1.505
1.500
1.495
1.490
1.485
1.480
1.475
1.470
1.465
1.460
1.455
-90
-80
-70
-60
-50
-40
-30
-20
-10
0
10
20
30
40
50

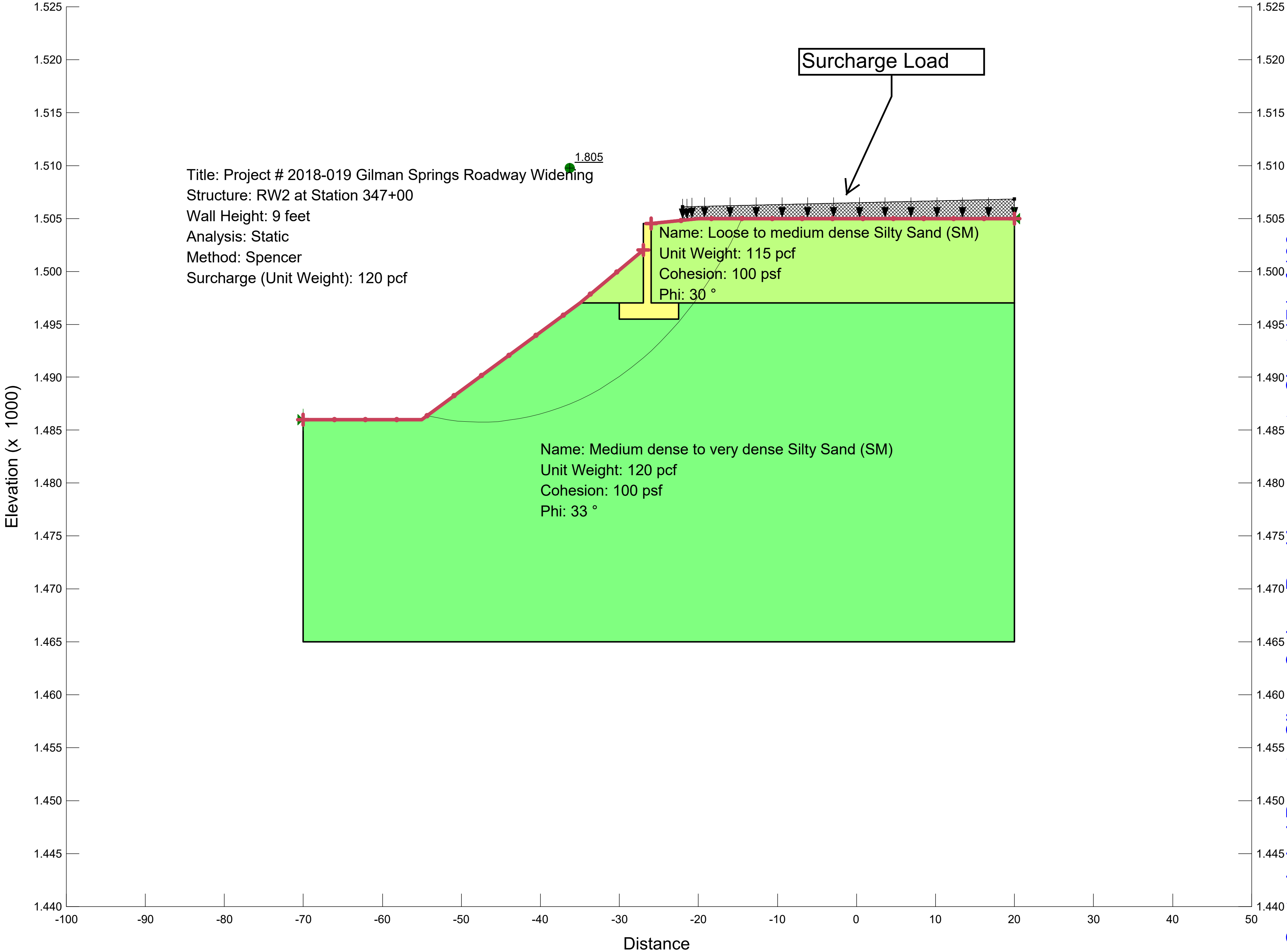

Title: Project # 2018-019 Gilman Springs Roadway Widening
Structure: RW1 at Station 344+00
Analysis: Temporary
Method: Spencer
Surcharge (Unit Weight): 120 pcf
1.378
Surcharge Load
Shoring
Name: Medium dense to dense Silty Sand (SM)
Unit Weight: 115 pcf
Cohesion: 100 psf
Phi: 30 °
Name: Medium dense to very dense Silty Sand (SM)
Unit Weight: 120 pcf
Cohesion: 100 psf
Phi: 33 °
Elevation (x 1000)
1.525
1.520
1.515
1.510
1.505
1.500
1.495
1.490
1.485
1.480
1.475
1.470
1.465
Distance
-90
-80
-70
-60
-50
-40
-30
-20
-10
0
10
20
30
40
50


Surcharge Load
1.683
Title: Project # 2018-019 Gilman Springs Roadway Widening
Structure: RW1 at Station 344+00.00
Wall Height: 10.5 feet
Analysis: Static
Method: Spencer
Surcharge (Unit Weight): 120 pcf
Name: Medium dense to dense Silty Sand (SM)
Unit Weight: 115 pcf
Cohesion: 100 psf
Phi: 30 °
Name: Medium dense to very dense Silty Sand (SM)
Unit Weight: 120 pcf
Cohesion: 100 psf
Phi: 33 °
Elevation (x 1000)
Distance
1.525
1.520
1.515
1.510
1.505
1.500
1.495
1.490
1.485
1.480
1.475
1.470
1.465
-90
-80
-70
-60
-50
-40
-30
-20
-10
0
10
20
30
40
50


1.051
Title: Project # 2018-019 Gilman Springs Roadway Widening
Structure: RW1 at Station 344+00.00
Wall Height: 10.5 feet
Analysis: Psuedo Static
Method: Spencer
Horz Seismic Load: 0.3
Name: Medium dense to dense Silty Sand (SM)
Unit Weight: 115 pcf
Cohesion: 100 psf
Phi: 30 °
Name: Medium dense to very dense Silty Sand (SM)
Unit Weight: 120 pcf
Cohesion: 100 psf
Phi: 33 °
Elevation (x 1000)
Distance
1.525
1.520
1.515
1.510
1.505
1.500
1.495
1.490
1.485
1.480
1.475
1.470
1.465
-90
-80
-70
-60
-50
-40
-30
-20
-10
0
10
20
30
40
50



Title: Project # 2018-019 Gilman Springs Roadway Widening
Structure: RW2 at Station 347+00
Analysis: Temporary
Method: Spencer
Surcharge (Unit Weight): 120 pcf
1.592
Surcharge Load
Shoring
Name: Loose to medium dense Silty Sand (SM)
Unit Weight: 115 pcf
Cohesion: 100 psf
Phi: 30 °
Name: Medium dense to very dense Silty Sand (SM)
Unit Weight: 120 pcf
Cohesion: 100 psf
Phi: 33 °
Elevation (x 1000)
Distance


Surcharge Load
1.805
Title: Project # 2018-019 Gilman Springs Roadway Widening
Structure: RW2 at Station 347+00
Wall Height: 9 feet
Analysis: Static
Method: Spencer
Surcharge (Unit Weight): 120 pcf
Name: Loose to medium dense Silty Sand (SM)
Unit Weight: 115 pcf
Cohesion: 100 psf
Phi: 30 °
Name: Medium dense to very dense Silty Sand (SM)
Unit Weight: 120 pcf
Cohesion: 100 psf
Phi: 33 °
Elevation (x 1000)
Distance
1.525
1.520
1.515
1.510
1.505
1.500
1.495
1.490
1.485
1.480
1.475
1.470
1.465
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1.445
1.440
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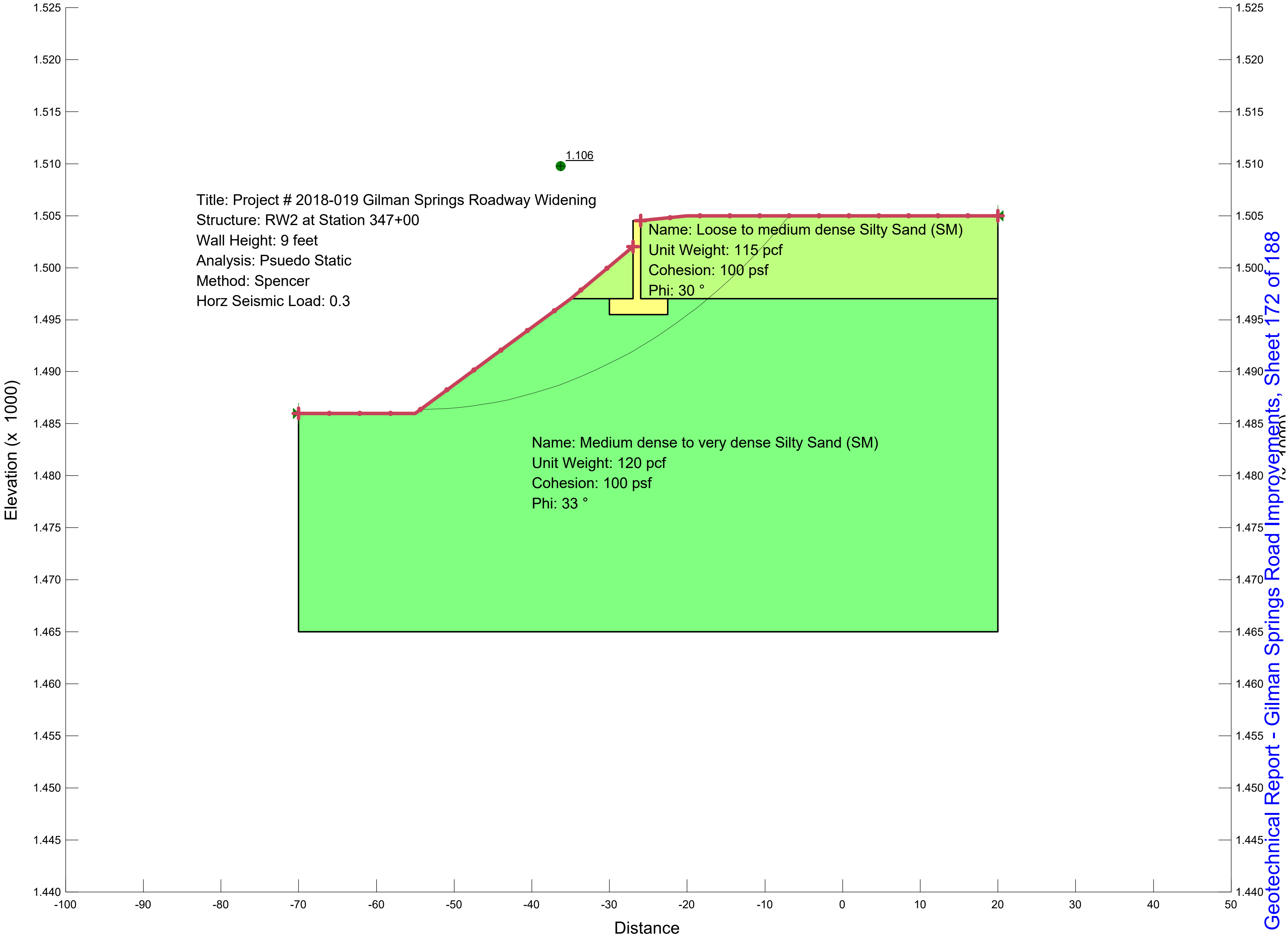
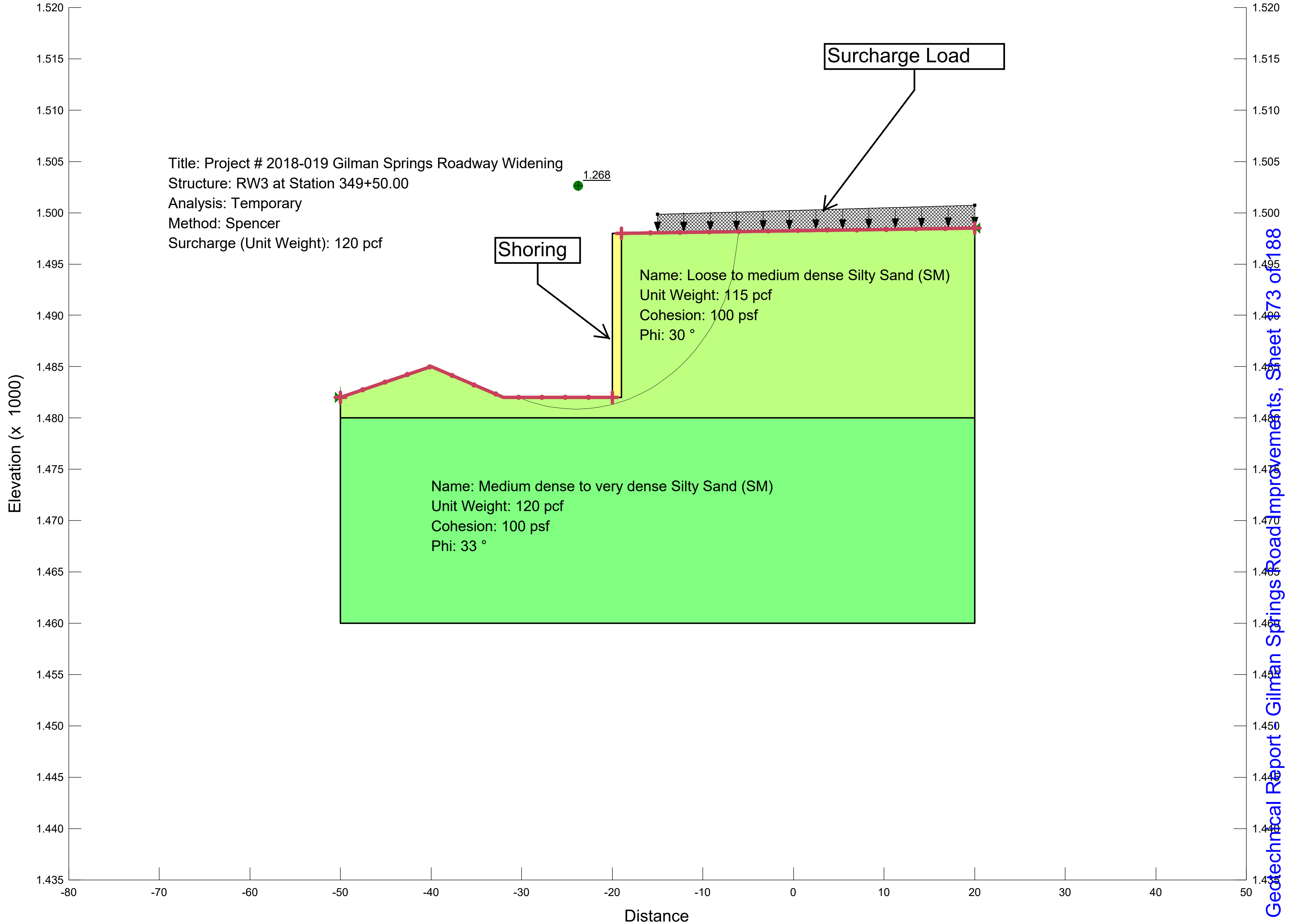
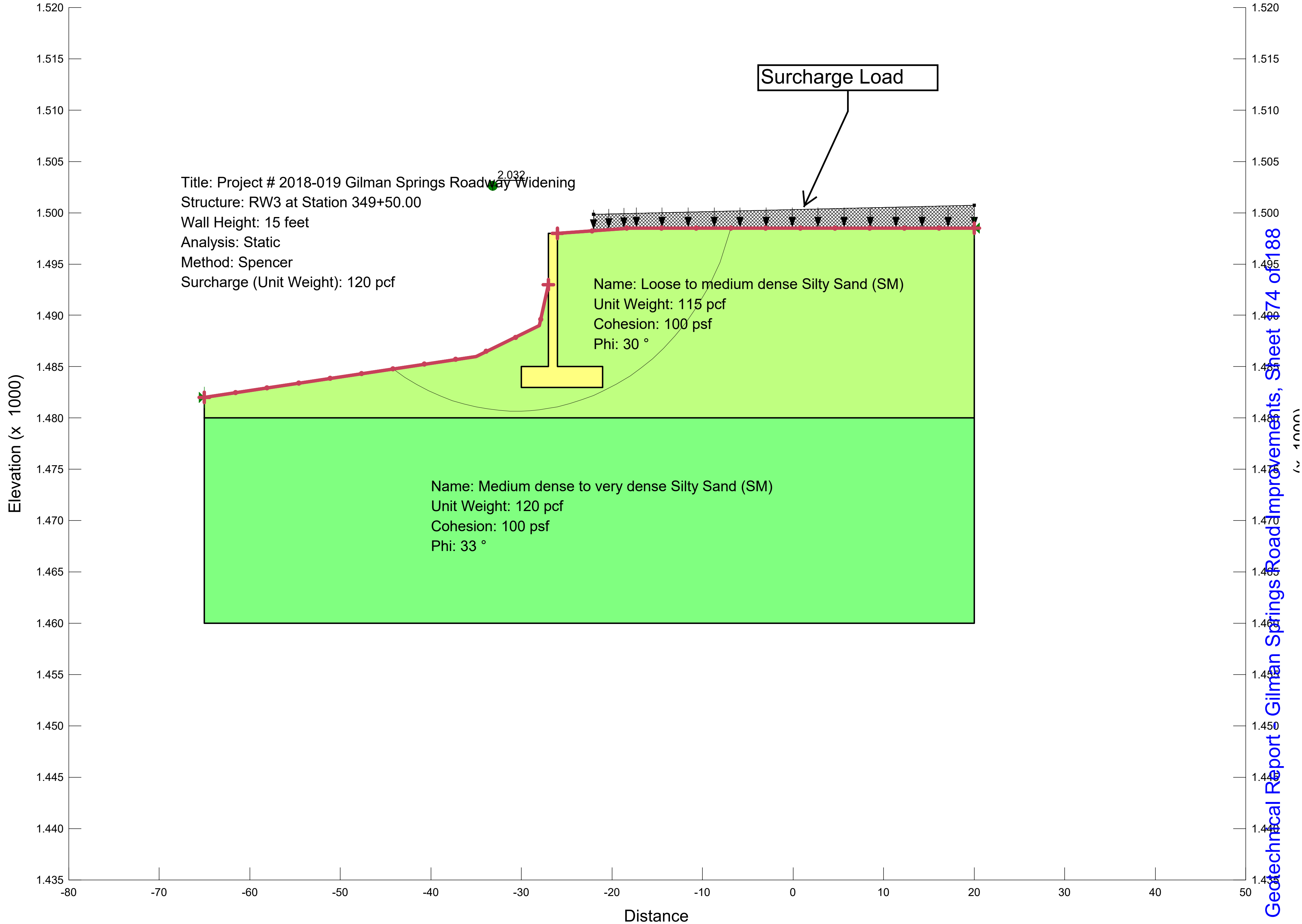

1.106
Title: Project # 2018-019 Gilman Springs Roadway Widening
Structure: RW2 at Station 347+00
Wall Height: 9 feet
Analysis: Psuedo Static
Method: Spencer
Horz Seismic Load: 0.3
Name: Loose to medium dense Silty Sand (SM)
Unit Weight: 115 pcf
Cohesion: 100 psf
Phi: 30 °
Name: Medium dense to very dense Silty Sand (SM)
Unit Weight: 120 pcf
Cohesion: 100 psf
Phi: 33 °
Elevation (x 1000)
Distance


Title: Project # 2018-019 Gilman Springs Roadway Widening
Structure: RW3 at Station 349+50.00
Analysis: Temporary
Method: Spencer
Surcharge (Unit Weight): 120 pcf
1.268
Surcharge Load
Shoring
Name: Loose to medium dense Silty Sand (SM)
Unit Weight: 115 pcf
Cohesion: 100 psf
Phi: 30 °
Name: Medium dense to very dense Silty Sand (SM)
Unit Weight: 120 pcf
Cohesion: 100 psf
Phi: 33 °
Elevation (x 1000)
Distance



Surcharge Load
2.032
Title: Project # 2018-019 Gilman Springs Roadway Widening
Structure: RW3 at Station 349+50.00
Wall Height: 15 feet
Analysis: Static
Method: Spencer
Surcharge (Unit Weight): 120 pcf
Name: Loose to medium dense Silty Sand (SM)
Unit Weight: 115 pcf
Cohesion: 100 psf
Phi: 30 °
Name: Medium dense to very dense Silty Sand (SM)
Unit Weight: 120 pcf
Cohesion: 100 psf
Phi: 33 °
Elevation (x 1000)
Distance
1.520
1.515
1.510
1.505
1.500
1.495
1.490
1.485
1.480
1.475
1.470
1.465
1.460
1.455
1.450
1.445
1.440
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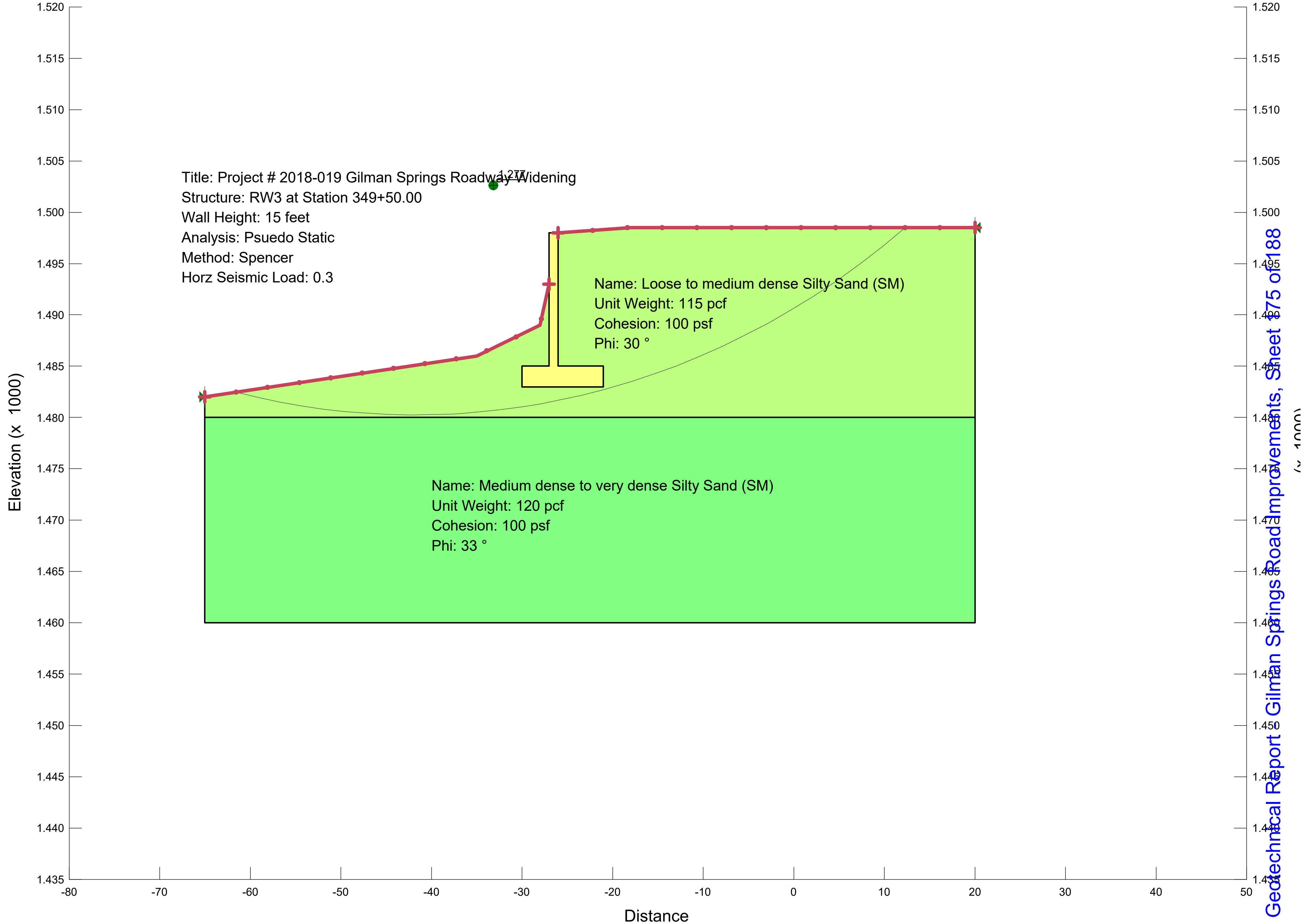
0
10
20
30
40
50



Title: Project # 2018-019 Gilman Springs Roadway Widening
Structure: RW3 at Station 349+50.00
Wall Height: 15 feet
Analysis: Psuedo Static
Method: Spencer
Horz Seismic Load: 0.3
1.277
Name: Loose to medium dense Silty Sand (SM)
Unit Weight: 115 pcf
Cohesion: 100 psf
Phi: 30 °
Name: Medium dense to very dense Silty Sand (SM)
Unit Weight: 120 pcf
Cohesion: 100 psf
Phi: 33 °
Elevation (x 1000)
Distance
1.520
1.515
1.510
1.505
1.500
1.495
1.490
1.485
1.480
1.475
1.470
1.465
1.460
1.455
1.450
1.445
1.440
1.435
-80
-70
-60
-50
-40
-30
-20
-10
0
10
20
30
40
50

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SHEET 1 OF 1
PROJECT NO. 2019-019
DATE 7/1/19
COMPUTED BY BH
CHECKED BY JB

PROJECT Gilman Springs Rd. Widening
SUBJECT Pavement Calcs (Rigid)

Assumptions

- One calc w/ Subgrade Type I w/ TI = 9.5
- One calc w/ subgrade Type II w/ TI = 9.5
- Caltrans Pavement Climate Region ⇒ Inland Valley
- TI = 9.5
- No lateral support

Subgrade Type I (TI=9.5) w/out lateral support

0.90′ JPCP ✓
0.60′ AB ✓

Subgrade Type II (TI=9.5) w/out lateral support

0.90′ JPCP ✓
1.00′ AB ✓

Table 623.1A
Relationship Between Subgrade Type(1)

Subgrade Type(2)	Unified Soil Classification System (USCS)
I	SC, SP, SM, SW, GC, GP, GM, GW
II	CH (PI ≤ 12), CL, MH, ML
III	CH (PI > 12)

NOTES:

(1) See Topic 614 for further discussion on subgrade and USCS.

(2) Choose more conservative soil type (i.e., use soil with a lower subgrade type) if native soil can be classified by more than one type.

Legend

PI = Plasticity Index

(4) Determine Whether Pavement Has Lateral Support Along Both Longitudinal Joints. The pavement is considered to have lateral support if any of the following exist:

- longitudinal joints are tied to an adjacent lane or shoulder,
- tied rigid shoulders are present, or
- a widened slab is present.

If lateral support is provided along only one longitudinal joint, then the pavement is considered to have no lateral support. As shown in Tables 623.1B through M, pavement thicknesses are reduced slightly for slabs engineered with lateral support along both longitudinal joints.

(5) Select Pavement Structure. Using the Traffic Index provided or calculated from the traffic projections, select the desired pavement structure from the list of alternatives provided.

Note that although the pavement structures listed for each Traffic Index are considered to be acceptable for the climate, soil conditions, and design life desired, they should not be considered as equal designs. Some designs will perform better than others, have lower maintenance/repair costs, and/or lower construction life-cycle costs. For these reasons, the rigid pavement structures in these tables cannot be used as substitutes for the pavement structures shown in approved contract plans.

Topic 624 – Engineering Procedures for Pavement Preservation

624.1 Preventive Maintenance

Examples of rigid pavement preventive maintenance strategies include the following or combinations of the following:

- Seal random cracks.
- Joint seal, repair/replace existing joint seals.
- Dowel bar retrofit.
- Grinding or grooving to maintain ride quality and/or restore surface texture.
- Special surface treatments (such as methacrylate, hardeners, and others).

Rigid pavement preventive maintenance strategies are discussed further in the Concrete Pavement Guide.

624.2 Capital Pavement Maintenance (CAPM)

A CAPM project is warranted if any of the following criteria is met:

(1) Continuously Reinforced Concrete Pavement

- Number of punchouts with high severity cracking is between 1 and 10 percent.

(2) Jointed Plain Concrete Pavement

- Number of slabs with 3rd stage cracking between 1 and 10 percent of a given travel lane-mile. Note, 3rd stage cracking is any slab with two or more intersecting cracks of at least ¾ inch in width.

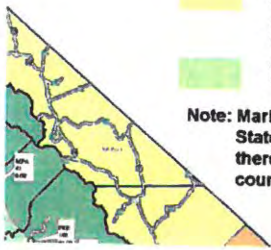
Caltrans

PAVEMENT CLIMATE REGIONS

October 5, 2005

- North Coast
- Central Coast
- Inland Valley
- Low Mountain
- High Mountain
- Desert
- High Desert
- South Coast
- South Mountain

Note: Markers indicate County/Route/Post Mile of State Hwys. at region boundaries. When there is no marker, the region follows a county boundary.



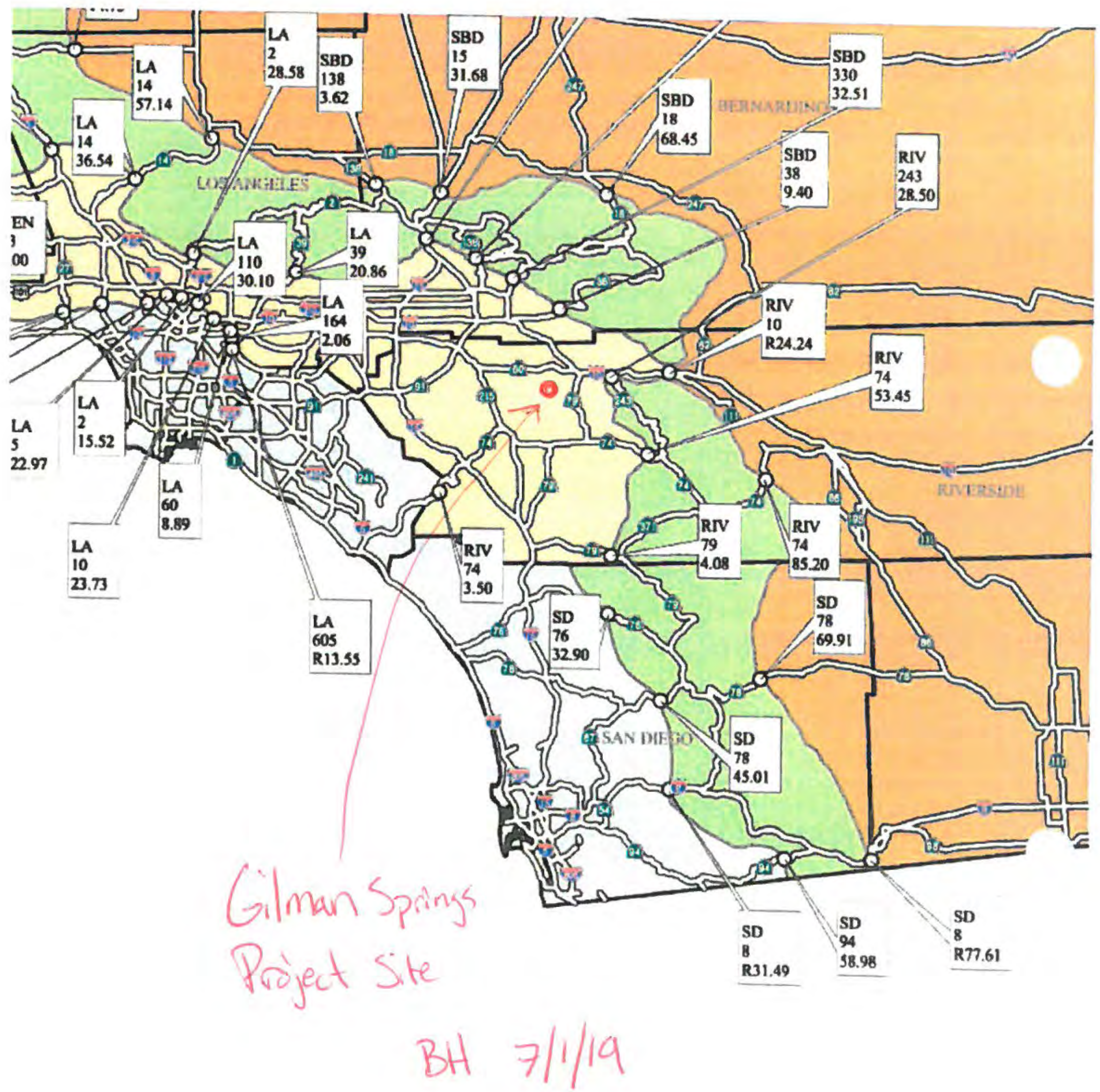
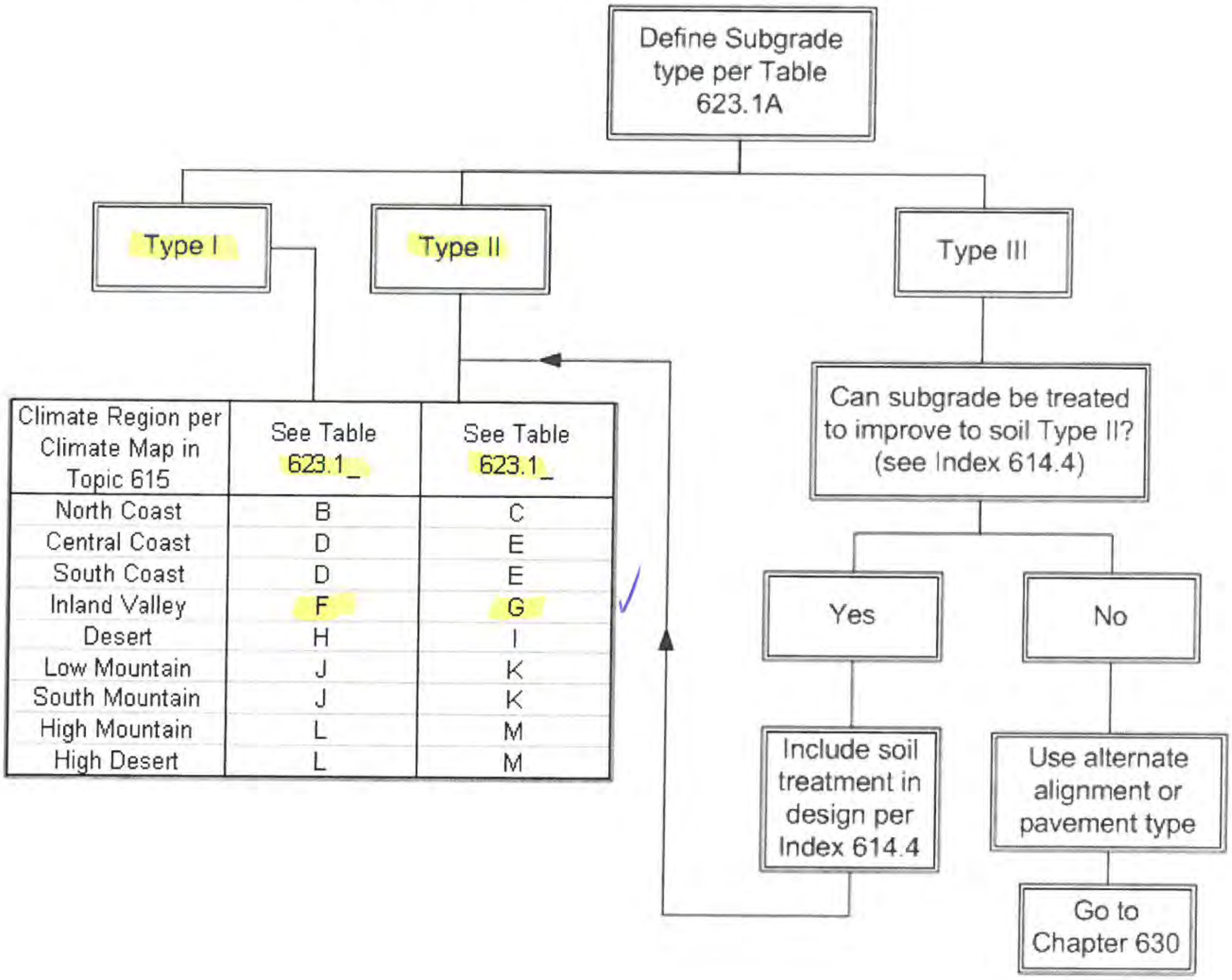


Figure 623.1

Rigid Pavement Catalog Decision Tree



Climate Region per Climate Map in Topic 615	See Table 623.1_	See Table 623.1_
North Coast	B	C
Central Coast	D	E
South Coast	D	E
Inland Valley	F	G
Desert	H	I
Low Mountain	J	K
South Mountain	J	K
High Mountain	L	M
High Desert	L	M

Table 623.1F
Rigid Pavement Catalog (Inland Valley, Type I Subgrade Soil) (1), (2), (3), (4), (5)

	Rigid Pavement Structural Depth					
TI	With Lateral Support (ft)			Without Lateral Support (ft)		
< 9			0.75 JPCP 0.50 AB			0.80 JPCP 0.50 AB
9.5 to 10			0.80 JPCP 0.60 AB			0.90 JPCP 0.60 AB
10.5 to 11	0.75 JPCP BB 0.35 LCB	0.75 JPCP 0.25 HMA-A	0.85 JPCP 0.70 AB	0.85 JPCP BB 0.35 LCB	0.90 JPCP 0.25 HMA-A	0.95 JPCP 0.70 AB
11.5 to 12	0.85 JPCP BB 0.35 LCB	0.85 JPCP 0.25 HMA-A	0.80 CRCP 0.25 HMA-A	0.95 JPCP BB 0.35 LCB	0.95 JPCP 0.25 HMA-A	0.85 CRCP 0.25 HMA-A
12.5 to 13	0.85 JPCP BB 0.35 LCB	0.90 JPCP 0.25 HMA-A	0.80 CRCP 0.25 HMA-A	1.00 JPCP BB 0.35 LCB	1.00 JPCP 0.25 HMA-A	0.90 CRCP 0.25 HMA-A
13.5 to 14	0.95 JPCP BB 0.35 LCB	0.95 JPCP 0.25 HMA-A	0.85 CRCP 0.25 HMA-A	1.05 JPCP BB 0.35 LCB	1.05 JPCP 0.25 HMA-A	0.95 CRCP 0.25 HMA-A
14.5 to 15	1.00 JPCP BB 0.35 LCB	1.00 JPCP 0.25 HMA-A	0.90 CRCP 0.25 HMA-A	1.15 JPCP BB 0.35 LCB	1.15 JPCP 0.25 HMA-A	1.00 CRCP 0.25 HMA-A
15.5 to 16	1.05 JPCP BB 0.35 LCB	1.05 JPCP 0.25 HMA-A	0.95 CRCP 0.25 HMA-A	1.20 JPCP BB 0.35 LCB	1.20 JPCP 0.25 HMA-A	1.05 CRCP 0.25 HMA-A
16.5 to 17	1.10 JPCP BB 0.35 LCB	1.10 JPCP 0.25 HMA-A	0.95 CRCP 0.25 HMA-A	1.25 JPCP BB 0.35 LCB	1.25 JPCP 0.25 HMA-A	1.10 CRCP 0.25 HMA-A
> 17	1.15 JPCP BB 0.35 LCB	1.15 JPCP 0.25 HMA-A	1.00 CRCP 0.25 HMA-A	1.30 JPCP BB 0.35 LCB	1.30 JPCP 0.25 HMA-A	1.10 CRCP 0.25 HMA-A

NOTES:

(1) Thicknesses shown for JPCP are for doweled pavement only. The thickness shown in these tables are not valid for nondoweled JPCP.

(2) Includes 0.03 ft sacrificial wearing course for future grinding of JPCP/CRCP.

(3) Portland cement concrete may be substituted for LCB when justified for constructibility or traffic handling. If Portland cement concrete is used in lieu of LCB, it must be placed in a separate lift than JPCP and must not be bonded to the JPCP.

(4) If ATPB is needed for TIs > 10.0 to perpetuate an existing treated permeable layer, place the ATPB between the surface layer (JPCP or CRCP) and the base layer. No deduction is made to the thickness of the base and subbase layers on account of the ATPB.

(5) Place a Bond Breaker between JPCP and LCB in all cases

LEGEND:

JPCP = Jointed Plain Concrete Pavement
CRCP = Continuously Reinforced Concrete Pavement
LCB = Lean Concrete Base
HMA-A = Hot Mix Asphalt (Type A)
ATPB = Asphalt Treated Permeable Base
AB = Class 2 Aggregate Base
TI = Traffic Index
BB = Base Bond Breaker

Table 623.1G
Rigid Pavement Catalog (Inland Valley, Type II Subgrade Soil)[(1), (2), (3), (4), (5)]

TI	Rigid Pavement Structural Depth					
	With Lateral Support (ft)			Without Lateral Support (ft)		
≤ 9			0.75 JPCP 1.00 AB			0.80 JPCP 1.00 AB
9.5 to 10			0.80 JPCP 1.00 AB			0.90 JPCP 1.00 AB
10.5 to 11	0.75 JPCP BB 0.35 LCB 0.60 AS	0.75 JPCP 0.25 HMA-A 0.60 AS	0.85 JPCP 1.30 AB	0.85 JPCP BB 0.35 LCB 0.60 AS	0.90 JPCP 0.25 HMA-A 0.60 AS	0.95 JPCP 1.30 AB
11.5 to 12	0.85 JPCP BB 0.35 LCB 0.60 AS	0.85 JPCP 0.25 HMA-A 0.60 AS	0.80 CRCP 0.25 HMA-A 0.60 AS	0.95 JPCP BB 0.35 LCB 0.60 AS	0.95 JPCP 0.25 HMA-A 0.60 AS	0.85 CRCP 0.25 HMA-A 0.60 AS
12.5 to 13	0.85 JPCP BB 0.35 LCB 0.70 AS	0.90 JPCP 0.25 HMA-A 0.70 AS	0.80 CRCP 0.25 HMA-A 0.70 AS	1.00 JPCP BB 0.35 LCB 0.70 AS	1.00 JPCP 0.25 HMA-A 0.70 AS	0.90 CRCP 0.25 HMA-A 0.70 AS
13.5 to 14	0.95 JPCP BB 0.35 LCB 0.70 AS	0.95 JPCP 0.25 HMA-A 0.70 AS	0.85 CRCP 0.25 HMA-A 0.70 AS	1.05 JPCP BB 0.35 LCB 0.70 AS	1.05 JPCP 0.25 HMA-A 0.70 AS	0.95 CRCP 0.25 HMA-A 0.70 AS
14.5 to 15	1.00 JPCP BB 0.35 LCB 0.70 AS	1.00 JPCP 0.25 HMA-A 0.70 AS	0.90 CRCP 0.25 HMA-A 0.70 AS	1.15 JPCP BB 0.35 LCB 0.70 AS	1.15 JPCP 0.25 HMA-A 0.70 AS	1.00 CRCP 0.25 HMA-A 0.70 AS
15.5 to 16	1.05 JPCP BB 0.35 LCB 0.70 AS	1.05 JPCP 0.25 HMA-A 0.70 AS	0.95 CRCP 0.25 HMA-A 0.70 AS	1.20 JPCP BB 0.35 LCB 0.70 AS	1.20 JPCP 0.25 HMA-A 0.70 AS	1.05 CRCP 0.25 HMA-A 0.70 AS
16.5 to 17	1.10 JPCP BB 0.35 LCB 0.70 AS	1.10 JPCP 0.25 HMA-A 0.70 AS	0.95 CRCP 0.25 HMA-A 0.70 AS	1.25 JPCP BB 0.35 LCB 0.70 AS	1.25 JPCP 0.25 HMA-A 0.70 AS	1.10 CRCP 0.25 HMA-A 0.70 AS
> 17	1.15 JPCP BB 0.35 LCB 0.70 AS	1.15 JPCP 0.25 HMA-A 0.70 AS	1.00 CRCP 0.25 HMA-A 0.70 AS	1.30 JPCP BB 0.35 LCB 0.70 AS	1.30 JPCP 0.25 HMA-A 0.70 AS	1.10 CRCP 0.25 HMA-A 0.70 AS

NOTES:

(1) Thicknesses shown for JPCP are for doweled pavement only. The thickness shown in these tables are not valid for nondoweled JPCP.

(2) Includes 0.03 ft sacrificial wearing course for future grinding of JPCP/CRCP.

(3) Portland cement concrete may be substituted for LCB when justified for constructibility or traffic handling. If Portland cement concrete is used in lieu of LCB, it must be placed in a separate lift than JPCP and must not be bonded to the JPCP.

(4) If ATPB is needed for TIs > 10.0 to perpetuate an existing treated permeable layer, place the ATPB between the surface layer (JPCP or CRCP) and the base layer. No deduction is made to the thickness of the base and subbase layers on account of the ATPB.

(5) Place a Bond Breaker between JPCP and LCB in all cases

LEGEND:

JPCP = Jointed Plain Concrete Pavement
CRCP = Continuously Reinforced Concrete Pavement
LCB = Lean Concrete Base
HMA-A = Hot Mix Asphalt (Type A)
BB = Base Bond Breaker
ATPB = Asphalt Treated Permeable Base
AB = Class 2 Aggregate Base
AS = Class 2 Aggregate Subbase
TI = Traffic Index

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SHEET ____ OF ____
PROJECT NO. 2018-019
DATE 7/11/19
COMPUTED BY BH
CHECKED BY OB

PROJECT Gilman Springs Rd.
SUBJECT Flexible Pavement Design Full Depth HMA

Assumptions

- Full depth HMA
- TI = 9.5
- Sub-grade RV=20

(1) $GE_{Full\ HMA} = 0.0032 \times TI \times (100 - R_{sub-grade}) + 0.10'$ ✓ (0.10' = Safety Factor)

$GE_{Full\ HMA} = 0.0032(9.5)(100-20) + 0.10 = \underline{2.532'}$ ✓

(2) From Table 633.1, closest GE_{HMA} to 2.532' is $GE_{HMA} = 2.51$ which corresponds to a full depth HMA of **t = 1.10 ft** ✓

Assumptions

- Full depth HMA
- TI = 9.5
- Sub-grade RV=50

(1) $GE_{Full\ HMA} = 0.0032 \times TI \times (100 - R_{subgrade}) + 0.10'$ ✓ (0.10' = Safety Factor)

$GE_{Full\ HMA} = 0.0032(9.5)(100-50) + 0.10' = 1.62'$ ✓

(2) From Table 633.1, closest GE_{HMA} to 1.62' is $GE_{HMA} = 1.64$ which corresponds to a full depth HMA of **t = 0.80 ft** ✓

appropriate G_f for that layer material, or from Table 633.1. The layer thickness determined by dividing GE by G_f is rounded up to the next higher value in 0.05-foot increments.

$$\text{Thickness (t)} = \frac{GE}{G_f}$$

The minimum thickness of any asphalt layer should not be less than three times the maximum aggregate size. Also, the minimum thickness of the dense graded HMA surface course should not be less than 0.15 foot. The limit thicknesses for placing HMA for each TI, and the limit thickness for each type of base and subbase materials are shown in Table 633.1

Base and subbase materials, other than ATPB, should each have a minimum thickness of 0.35 foot. When the calculated thickness of base or subbase material is less than the desired 0.35 foot minimum thickness, either: (a) increase the thickness to the minimum without changing the thickness of the overlying layers, or (b) eliminate the layer and increase the thickness of the overlying layers to compensate for the reduction in GE.

Generally, the layer thickness of Lime Stabilized Soil (LSS) and Cement Stabilized Soil (CSS) should be limited with 0.65 foot as the minimum and 2 feet as the maximum. A surface layer placed directly on the LSS or CSS should have a thickness of at least 0.25 foot.

The thicknesses determined by the procedures outlined in this section are not intended to preclude other combinations and thicknesses of materials. Adjustments to the thickness of the various materials may be made to accommodate construction restrictions or practices, and minimize costs, provided the minimum thicknesses, maximum thicknesses, and minimum GE requirements (including safety factors) of the entire pavement structure and each layer are as specified.

Whereas the empirical method and Table 633.1 do not provide for RHMA-G material, it is possible to substitute the top 0.15 to 0.20 foot of the design HMA thickness with an equal thickness of RHMA-G.

(2) *Procedures for Full Depth Hot Mix Asphalt.* Full depth hot mix asphalt applies when the pavement structure is comprised entirely of a flexible surface layer in lieu of base and subbase. The flexible surface layer may be comprised of a single or multiple types of flexible pavements including HMA, RHMA, interlayers, special asphalt binders, or different mix designs. Considerations regarding worker safety, short construction windows, the amount of area to be paved, or temporary repairs may make it desirable in some instances to reduce the total thickness of the pavement by placing full depth hot mix asphalt. Full depth hot mix asphalt also is less affected by moisture or frost, does not let moisture build up in the subgrade, provides no permeable layers that entrap water, and is a more uniform pavement structure. Use the standard equation in Index 633.1(1) with the California R-value of the subgrade to calculate the GE for the entire pavement structure based on TI and the subgrade R-value. Increase this GE by adding the safety factor of 0.10 foot to obtain the required GE for the flexible pavement. Then refer to Table 633.1, select the closest layer thickness for conventional hot mix asphalt, and determine the adjusted GE that it provides. The GE of the safety factor is not removed in this design. Adjust the final thickness as needed when using other types of materials than hot mix asphalt. The top 0.15 to 0.2 foot of the HMA thickness can be substituted with an equal thickness of RHMA-G.

A Treated Permeable Base (TPB) layer may be placed below full depth hot mix asphalt on widening projects to perpetuate or match, an existing TPB layer for continuity of drainage. Reduce the GE of the surface layer by the amount of GE provided by the TPB. In no case should the initial GE of the surface layer over the TPB be less than 40 percent of the GE required over the subbase as calculated by the standard engineering equation. When there is

Table 633.1
Gravel Equivalents (GE) and Thickness of Structural Layers (ft)

Actual Layer Thickness (ft) [5]	HMA[(1), (2)]											Base and Subbase [(3), (4)]					
	Traffic Index (TI)											TI is not a factor					
	5.0 & below	5.5 – 6.0	6.5 – 7.0	7.5 – 8.0	8.5 – 9.0	9.5 – 10.0	10.5 – 11.0	11.5 – 12.0	12.5 – 13.0	13.5 – 14.0	14.5 – 15.0	LCB	CTPB: CTB (Cl. A)	ATPB	CTB (Cl. B)	AB	AS
	G_f (For HMA thickness equal to or less than 0.5 ft, G_f decreases with TI) [(6)]											G_f (Constant for any base or subbase material irrespective of TI or thickness)					
	2.54	2.32	2.14	2.01	1.89	1.79	1.71	1.64	1.57	1.52	1.46	1.9	1.7	1.4	1.2	1.1	1.0
	GE for HMA layer (ft)											GE for Base or Subbase layer (ft)					
0.10	0.25	0.23	0.21	0.20	0.19	0.18	0.17	0.16	0.16	0.15	0.15	--	--	--	--	--	--
0.15	0.38	0.35	0.32	0.30	0.28	0.27	0.26	0.25	0.24	0.23	0.22	--	--	--	--	--	--
0.20	0.51	0.46	0.43	0.40	0.38	0.36	0.34	0.33	0.31	0.30	0.29	--	--	--	--	--	--
0.25	0.63	0.58	0.54	0.50	0.47	0.45	0.43	0.41	0.39	0.38	0.37	--	--	0.35	--	--	--
0.30	0.76	0.69	0.64	0.60	0.57	0.54	0.51	0.49	0.47	0.45	0.44	--	--	0.42	--	--	--
0.35	0.89	0.81	0.75	0.70	0.66	0.63	0.60	0.57	0.55	0.53	0.51	0.67	0.60	0.49	0.42	0.39	0.35
0.40	1.01	0.93	0.86	0.80	0.76	0.72	0.68	0.65	0.63	0.61	0.59	0.76	0.68	0.56	0.48	0.44	0.40
0.45	1.14	1.04	0.96	0.90	0.85	0.81	0.77	0.74	0.71	0.68	0.66	0.86	0.77	0.63	0.54	0.50	0.45
0.50	1.27	1.16	1.07	1.00	0.94	0.90	0.85	0.82	0.79	0.76	0.73	0.95	0.85	0.70	0.60	0.55	0.50
0.55	1.41	1.29	1.19	1.12	1.05	1.00	0.95	0.91	0.87	0.84	0.81	1.05	0.94	0.77	0.66	0.61	0.55
0.60	1.58	1.45	1.34	1.25	1.18	1.12	1.07	1.02	0.98	0.95	0.91	1.14	1.02	0.84	0.72	0.66	0.60
0.65	1.76	1.61	1.49	1.39	1.31	1.25	1.19	1.14	1.09	1.05	1.02	1.24	1.11	0.91	0.78	0.72	0.65
0.70	--	1.78	1.64	1.54	1.45	1.38	1.31	1.26	1.21	1.16	1.12	1.33	1.19	--	0.84	0.77	0.70
0.75	--	1.95	1.80	1.69	1.59	1.51	1.44	1.38	1.32	1.27	1.23	1.43	1.28	--	0.90	0.83	0.75
0.80	--	2.12	1.96	1.84	1.73	1.64	1.57	1.50	1.44	1.39	1.34	1.52	1.36	--	0.96	0.88	0.80
0.85	--	--	2.13	1.99	1.88	1.78	1.70	1.63	1.56	1.51	1.46	1.62	1.45	--	1.02	0.94	0.85
0.90	--	--	2.30	2.15	2.03	1.92	1.83	1.76	1.69	1.63	1.57	1.71	1.53	--	1.08	0.99	0.90
0.95	--	--	--	2.31	2.18	2.07	1.97	1.89	1.81	1.75	1.69	1.81	1.62	--	1.14	1.05	0.95
1.00	--	--	--	2.47	2.33	2.21	2.11	2.02	1.94	1.87	1.81	1.90	1.70	--	1.20	1.10	1.00
1.05	--	--	--	2.64	2.49	2.36	2.25	2.16	2.07	2.00	1.93	2.00	1.79	--	1.26	1.16	1.05
1.10	--	--	--	--	2.65	2.51	2.40	2.29	2.20	2.12	2.05	--	--	--	--	--	1.10
1.15	--	--	--	--	2.81	2.67	2.54	2.43	2.34	2.25	2.18	--	--	--	--	--	1.15
1.20	--	--	--	--	2.98	2.82	2.69	2.58	2.48	2.39	2.30	--	--	--	--	--	1.20
1.25	--	--	--	--	--	2.98	2.84	2.72	2.61	2.52	2.43	--	--	--	--	--	1.25
1.30	--	--	--	--	--	3.14	2.99	2.87	2.75	2.65	2.56	--	--				1.30
1.35	--	--	--	--	--	3.30	3.15	3.01	2.90	2.79	2.70	--	--				
1.40	--	--	--	--	--	--	3.31	3.16	3.04	2.93	2.83	--	--				
1.45	--	--	--	--	--	--	3.46	3.32	3.19	3.07	2.97	--	--				
1.50	--	--	--	--	--	--	3.62	3.47	3.33	3.21	3.10	--	--				
1.55	--	--	--	--	--	--	--	3.62	3.48	3.36	3.24	--	--				
1.60	--	--	--	--	--	--	--	3.78	3.63	3.50	3.38	--	--				
1.65	--	--	--	--	--	--	--	3.94	3.79	3.65	3.52	--	--				
1.70	--	--	--	--	--	--	--	--	3.94	3.80	3.67	--	--				
1.75	--	--	--	--	--	--	--	--	4.09	3.95	3.81	--	--				
1.80	--	--	--	--	--	--	--	--	4.25	4.10	3.96	--	--				
1.85	--	--	--	--	--	--	--	--	--	4.25	4.10	--	--				
1.90	--	--	--	--	--	--	--	--	--	4.40	4.25	--	--				
1.95	--	--	--	--	--	--	--	--	--	4.56	4.40	--	--	--	--	--	--
2.00	--	--	--	--	--	--	--	--	--	--	4.55	--	--	--	--	--	--

Full depth for RV = 30 based on GE = 2.23 and GE = 2.21 is the closest value in table to 2.23.
t = 1.0 inches

NOTES:
(1) Open Graded Friction Course (conventional and rubberized) is a non-structural wearing course and provides no structural value.
(2) Top portion of HMA surface layer (maximum 0.20 ft.) may be replaced with equivalent RHMA-G thickness. See Topic 631.3 for additional details.
(3) See Table 663.3 for additional information on Gravel Factors (G_f) and California R-values for base and subbase materials.
(4) When using Hot Mix Asphalt Base (HMAB), the HMAB is considered as part of the HMA layer. Therefore, the HMAB will be assigned the same G_f as the remainder of the HMA in the pavement structure.
(5) For HMA layer, select TI range, then go down to the appropriate GE and across to the thickness column. For base and subbase layer, select material type, then go down to the appropriate GE and across to the thickness column.
(6) These G_f values are for TIs shown and HMA thickness equal to or less than 0.5 foot only. For HMA thickness greater than 0.5 foot, appropriate G_f should be determined using the equation in Index 633.1(1)(d).

APPENDIX F -
RESPONSE TO REVIEW COMMENTS

RIVERSIDE COUNTY: PRELIMINARY GEOTECHNICAL REPORT COMMENTS

No.	Comment By:	Comments	Response/Actions	Response By:
1	Elmer Datuin	Section 1. The proposed project is to add one 12-ft wide lane in each direction, 8-ft wide paved shoulders, and a 2-ft wide painted median. That is a total of 34-ft wide addition to the existing. Is this the intent?	According to the scope of work project description, the improvements include widening to allow for 12-ft wide lanes in each direction, 8-ft wide paved shoulders and 2-ft wide median. There is no intent to widen 34 feet.	SN/BH
2	Elmer Datuin	Section 4.3.1. Add the use of cement or lime as a countermeasure for yielding or pumping soil. The use of geogrid or other geosynthetic products should also be considered in case this work activity is in a critical path where delay is not an option	Yes, we added a paragraph about use of cement/lime treatment of basement soils and the use of geotextile and geogrid when poor subgrade is encountered	SN/BH
3	Elmer Datuin	Section 4.8. As an option to the pavement sections listed, consider adding cement treated basement soil. Adding about 4-5% cement as a stabilizing agent to the soil could increase the R-value up to a max of 40. Using 30 R-value for the analysis, this yields a pavement section of 6" HMA (or 2" RHMA/4" HMA) over 13" AB. From a cost standpoint, this alternative section provides a savings to the overall project cost.	Yes, we added an optional pavement section (with cement treated basement soil) to the pavement section table. We provided a pavement structural section based on R-Value of 30.	SN/BH
4	Elmer Datuin	Section 4.8. Convert all pavement section thicknesses to feet.	Yes, we converted from inches to feet.	SN/BH
5	Elmer Datuin	Appendix D. Provide calculations for pavement section design.	Yes, pavement calculations were added in the "Calculation" appendix.	SN/BH
6	Alfredo Martinez	Comments were made with pen on physical copy of Report	Comments were addressed over a telephone conversation on September 13, 2019 with Alfredo (RC), Niranjan (DYA) and Britton (DYA)	SN/BH

RIVERSIDE COUNTY: PRELIMINARY GEOTECHNICAL REPORT COMMENTS

7	Vian Ghazi	We had to make some changes to our project description which will carry over to the Geotech report. On page 3 of the attached PDF I included a suggested "replacement" paragraph that includes the latest project info. I also noticed some odd characters on Appendix F, it looks like a weird print error that should hopefully be an easy fix.	Yes, we changed the Project description to match what you have given. We have also fixed the character problem in Appendix F.	SN/BH

DISTRIBUTION

1 copy: Mr. Michael Crull, PE
NCM Engineering Corporation
4740 Green River Road, Suite 218
Corona, CA 92880

QUALITY CONTROL REVIEWER

Saroj Weeraratne, PhD, PE, GE
Associate Engineer

BH/SN:dr