GEOTECHNICAL INVESTIGATION
FOR
LYON TANK ACCESS ROAD LANDSLIDE REPAIR
365 Madrone Drive
Boulder Creek, California

Prepared For
SAN LORENZO VALLEY WATER DISTRICT
Boulder Creek, California

Prepared By
HARO, KASUNICH AND ASSOCIATES, INC.
Geotechnical & Coastal Engineers

Project No. SC4090.1
August 2018

Project No. SC4090.1
6 August 2018

SAN LORENZO VALLEY WATER DISTRICT
13060 Highway 9
Boulder Creek, California 95006

Attention: Mr. Rick Rogers

Subject: Geotechnical Investigation

Reference: Lyon Tank Access Road Landslide Repair
365 Madrone Drive
Boulder Creek, California

Dear Mr. Rogers:

In accordance with the request of the San Lorenzo Valley Water District (SLVWD), Haro, Kasunich and Associates, Inc. (HKA) have performed a Geotechnical Investigation for the repair of the access road that services the Lyon Tank in Boulder Creek, California.

The accompanying report presents our conclusions and recommendations, as well as the results of the geotechnical investigation on which they are based. A broad soil mass disconnected from the hillside during the winter rain season of 2016/2017 and mobilized downslope leaving a large head scarp that undermined a portion of the access road including Madrone Road. The access road that services the subject water tank crosses over the soil mass in several locations. Portions of the road mobilized along with the soil mass in some locations and in other locations the road was completely buried.

The San Lorenzo Valley Water District (SLVWD) has requested that HKA develop an understanding of the unstable broad soil mass and present geotechnical recommendations for stabilization and reconstruction of the damaged portions of the access road. To better understand the geologic and geotechnical parameters of the project site, HKA completed a field exploration program that included, site reconnaissance, 16 test borings drilled to depths of 7.0 and 51.5 feet below the ground surface (bgs), and laboratory testing for mechanical properties of soil samples collected from within the test borings. The study area was topographically mapped several times by Professional Land Surveyor Paul Jensen. The soil mass continued to mobilize between surveys with most recent map dated February 2018.

Geologic sections were developed using the topographical map along with data collected during the field exploration. A worst case slope stability model of the hillside was created in cross section view by assigning mechanical properties (strength, density, moisture) to the soils layers in the geologic section. The slope stability analysis was completed with the aid of the computer software program SLOPE/W by GEOSLOPE. A double check of the inputs for the model was completed by back calculating the landslide that already occurred under wet winter conditions without the influence of seismic shaking.

The preliminary results of the analysis were presented to the representatives of the SLVWD. In brief a broad soil mass has disconnected from the hillside from the head scarp down to Hessey Creek. The disconnected soil mass is unstable under wet winter conditions without seismic shaking and will continue to reactivate overtime and creep downslope. Although the entire disconnected soil mass could be stabilized, it would be more practical to stabilize just the soil mass starting from the outboard side of Madrone Road upslope to the head scarp.

HKA recommends unloading the soil mass by removing the upper 5 (+/-) feet of soil starting just below Madrone Road up to the head scarp. The soil mass from Madrone Road up to the head scarp should be stabilized using two rows of buried secant piles. The lower row of secant piles would be constructed along the outboard side of Madrone Road and is estimated to be 200 feet long by as much as 55 feet deep. The upper row of secant piles is recommended to be constructed on the hillside approximately half way up to the head scarp from Madrone Road. The upper row is estimated to be 150 feet long by as much as 40 feet deep. To rebuild and secure the severely damaged portion of the upper access road where the soil mass dislodged from the head scarp an engineered fill slope is recommended to be constructed from the upper row of secant piles up to the inboard side of the damaged upper road.

HKA re-iterates that the disconnected soil mass downslope from Madrone Road is unstable and the recommended rows of secant piles presented in this report will not stabilize this portion of the hillside. Furthermore, any surcharge placed upon the soil mass downslope from Madrone Road may exacerbate instability. If SLVWD would like to eliminate re-activation of the disconnected soil mass below Madrone Road please communicate this objective with HKA. We anticipate a temporary road will need to be constructed to install the upper row of secant piles on the hillside. Additional working meetings with structural and civil designers, specialty contractors, and SLVWD are anticipated, in order to develop viable working drawings.

If you have any questions concerning the data or conclusions presented in this report, please call our office.

Respectfully Submitted,

HARO, KASUNICH AND ASSOCIATES, INC.

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GEOTECHNICAL INVESTIGATION

1. Introduction

This report presents the findings, conclusions and recommendations of our Geotechnical Investigation for the Lyon Tank Access Road Landslide Repair Project. The Tank site is located at the end of Madrone Drive in Boulder Creek, California (see Site Vicinity Map, Figure 1 in Appendix A). A broad soil mass disconnected from the hillside and mobilized downslope. We will refer to the disconnected soil mass as the “landslide” from here forward. The slow moving landslide, which initially activated in the winter of 2017, has resulted in significant damage to the only access road to the SLVWD Lyon Water Tank and Water Treatment Facility. The water tank is the main water supply for residents within the San Lorenzo Valley Water district. The landslide is located between the upper most road that provides access to the base of the Lyon Tank which we will refer to as the “upper road” (that traverses the head scarp) and Hessey Creek, located about 200 feet downslope and to the east. A 160 foot long portion of Madrone Road which we will refer to as the “lower road” crosses the active landslide deposit and has been damaged. This report presents the results of our field investigations, laboratory testing, static and seismic slope stability analysis, and development of geotechnical design criteria and recommendations for stabilization of a select but imperative portion of the landslide.

Survey Maps with cross sections of the landslide area were prepared by Paul Jensen, and provided for our use. The landslide maps, with cross sections, are dated February 2017, June 2017, October 2017, and February 2018. The landslide area was surveyed four times to assist in evaluating the movement of the active landslide and to define potential toe of slip surfaces. The locations of exploratory borings indicated on the maps were surveyed by Mr. Jensen. The ground surface elevations at each boring location on the landslide deposit vary depending on the map date due to the ongoing movement of the landslide.

The Lyon Tank lower road crosses the landslide site immediately before a hairpin turn up to the tank. Just beyond the hairpin turn, the road forks. The lower fork of the road or the “upper road” leading to the tank has been damaged and is unusable due to landsliding. Before the hairpin, a 160 foot length of the lower road has been damaged by landsliding and temporarily repaired. The initial movement of the landslide was first observed by Haro Kasunich and Associates, Inc. (HKA) on 13 February 2017 during an on-site meeting with SLVWD Operations Management staff. We were informed ground and asphalt cracks were first observed in January 2017 after heavy rainfall at the site. At the time of our 13 February visit, the west lateral edge of the landslide and access road had dropped 2” to 4” and a 2’ to 3’ wide asphalt patch had been placed and compacted from the north to south side of the road to bridge the damaged area. The patch covered over a zone of 1” to 2” wide cracks in the asphalt. Soil cracks with a few inches of vertical displacement extended up the slope toward the upper access road.

A 15 inch diameter culvert on the surface of the slope below the access road on the west side of the slide was observed to be discharging water and angular gravel. The gravel was part of a gravel blanket drain installed during grading for construction of the access road to the Water Treatment Plant. The landslide movement dislodged and broke the pipe, allowing the gravel to flow into the culvert and then to be discharged out the end of the culvert.

In addition to the access road landslide, surficial sliding on the upper slope between the Lyon Tank and Water Treatment Plant was first observed by HKA on 13 February 2017. The slumps occurred about mid slope in several areas. On 15 February, the portion of the upper slope where slump slides occurred was covered with plastic sheeting and sandbags tied by rope to anchor the plastic and divert incident rainfall from the slope to the asphalt road below.

The access road landslide continued to move after heavy rainfall and by 22 February the east side of the upper access road down dropped several inches and numerous 1" to 2" wide cracks along a 50 foot long portion of the road had developed as the slide moved downslope. By Sunday 26 February, the landslide moved significantly and a 70' long portion of the road collapsed at the top of the landslide. The landslide left a 1' to 5' high head scarp at the inboard side of the lower of the upper roads. The west end of the access road dropped about 4 feet and subsurface water was emanating from the landslide scarp at the access road. Buckling of the pavement was observed on the downslope portion of the access road crossing the landslide. In early March, the entire landslide surface from the access road to the slide head scarp and side scarps was covered with plastic sheeting and rope tied sandbags to prevent incident rainfall from infiltrating into the covered part of the landslide deposit.

Several large trees on the landslide deposit were observed to be leaning significantly and posed a danger to the field investigation. The district hired a tree service to remove the worst of the leaning trees, which were removed in March and/or April 2017. On the west side of the access road, which had dropped about 6 feet, the district built a temporary gravel fill slope to provide vehicle access to the Water Treatment Plant and Lyon Tank for workers who perform daily maintenance and monitoring duties required to continue supplying potable water to District customers.

The movement of the landslide continued until early May 2017 when our initial borings were drilled. The plastic sheeting had been removed prior to our drilling and the landslide was re-surveyed in May. At that time the west side and the upper portion of the landslide had dropped from 6' to 8' and a bulge had developed on the slope between the creek and the access road. The west side of the section of the access road crossing the landslide had dropped 6' to 7'. The east side of the access road on the landslide had buckled due to uplift pressure from the slide and the curb drain inlet on the inboard side of the road was damaged by the landslide. The east side of the slide is buttressed by a previous road repair in 1986 which replaced a failed wood crib wall. The repair consisted of removal of soil on the slope and in the stream channel, installation of a large culvert in the stream, and placement and compaction of rock and soil backfill on the slope and road.

After our initial borings, a path was cleared on the slope below the access road to provide access for a drill rig to advance an additional 4 borings on the landslide deposit below the access road and 1 boring on the landslide deposit above the access road. Adjacent to Boring B-10 on the slope between the access road and Hessey Creek, a constant flow of water seeping from the toe of a steep slope was observed.

A fourth survey of the site in October 2017 indicates the upper landslide headscarp had increased to 6' to 10' high and the landslide had moved up to 4 feet horizontally toward the creek since the first survey (which had been done after significant movement had already occurred).

New longitudinal cracks in the upper road to the Water Treatment Plant were reported by the district in late October 2017. The cracks on the upper Water Treatment Plant parking area were generally 1/32" to 1/16" wide. One asphalt crack was 1/2" wide. We returned to the site and drilled 4 supplemental borings in the Water Treatment Plant parking area to assess the subsurface conditions underlying the parking area and the slope descending to the Lyon Tank.

Based on geological review of published regional geologic maps of the area, we found a fault zone traverses through the project area. The historical presence of the fault zone in the area likely sheared and weakened the earth materials during geologic time and likely also disrupted groundwater flow. The landslide slip surface has extraordinarily weak earth materials along it with very low residual strengths; in part because of historical shearing during previous instability including the 2017 re-activation. The above factors complicate landslide repair because of difficulty in maintaining safety during any mass excavation of the landslide materials. The landslide mass is expected to continue to be unstable and may expand should nothing be done to mitigate the existing condition.

HKA performed field explorations (test borings); 1) to profile the subsurface earth materials; 2) obtain samples; and 3) perform a laboratory testing program. On September 15th, 2017, a memorandum was prepared by HKA including discussion about slope improvement feasibilities. In this report, we present results of the geotechnical analysis which is limited to the 2017 landslide. The proposed mitigation solution is to install two rows of secant piles, one along the outboard side of the lower road and another on the hillside midway upslope to the upper road. The two rows of piles should be advanced into bedrock a minimum of 15 feet. A temporary road will need to be graded to install the upper row of secant piles. The upper road is recommended to be re-contructed by grading an engineered fill slope with a slope gradient of 2H:1V with its toe at the upper row of secant piles and crest along the outboard board side of the upper road. To reconstruct the travel way of the upper road, the fill slope would continue at 2%-5% from its crest to the inboard cut slope along the upper road.

2. Purpose and Scope

Our scope of services included review of existing geotechnical and geologic information related to the site, drilling and sampling in sixteen (16) exploratory borings, laboratory testing, and engineering analysis. The key focus was evaluation of the unstable landslide mass using the projected failure mode geometry; and evaluation of a practical method to improve the slope. The purpose of these services is to provide information and geotechnical recommendations relative to:

- Subsurface soil conditions;
- Groundwater conditions;
- Seismic considerations;
- Relative stability of landslide deposits and in-situ earth materials within the slip-out area (under static loading conditions);
- Earthwork recommendations.

3. Field Exploration and Laboratory Testing

3.1. Field Exploration

The field investigation has been completed at site by drilling 16 boreholes over a period of approximately 6.5 months. Twelve boreholes were drilled at the 2017 landslide between the Lyon Tank and the existing creek at the base of the slope. Boreholes 13 to 16 were drilled at the top of the slope south of the Lyon tank. B-1 to B-12 were drilled within the landslide area. The specifics of the drilled boreholes are presented in Table 1.

Table 1
Drilled Boreholes Specification

BH No.	BH Drilling Date	BH Depth (ft)	Approximate BH Top Elevation (ft)	Approximate BH Bottom Elevation (ft)
B-1	May 4, 2017	51.5	819.0	767.5
B-2	May 4, 2017	31.5	819.5	788.0
B-3	May 23, 2017	36.5	815.5	779.0
B-4	May 23, 2017	46.5	822.5	776.0
B-5	May 24, 2017	41.5	847.5	806.0
B-6	May 24, 2017	33.0	851.5	818.5
B-7	May 24, 2017	30.0	810.8	780.8
B-8	July 24, 2017	46.5	797.9	751.4
B-9	July 24, 2017	41.5	792.5	751.0
B-10	July 25, 2017	35.0	778.3	743.3
B-11	July 25, 2017	35.0	779.7	744.7
B-12	July 25, 2017	32.5	838.3	805.8
B-13	November 22, 2017	32.5	888.5	856.0
B-14	October 22, 2017	7.0	888.5	881.5
B-15	November 22, 2017	31.5	888.0	856.5
B-16	November 22, 2017	21.5	888.5	867.0

Representative soil samples were obtained from the exploratory borings at selected depths, or at major strata changes. These samples were recovered using a 3.0 inch O.D. Modified California Sampler (L), or by a 2.0 inch O.D. Standard Terzaghi Sampler (T). The SPT blow counts with large sampler (NL) should be reduced by a specific reduction factor to convert to Standard SPT blow counts (NS). The correlation between these two values are presented below:

$$N_S = N_L \, [(WH)/(623N \cdot 0.762m)] \, [(50.8^2-34.9^2)/(D_o^2-D_i^2)] \qquad \text{(Equation 1)}$$

The penetration blow counts noted on the boring logs were obtained by driving a sampler into the soil with a 140-pound hammer dropping through a 30-inch fall. The sampler was driven up to 18 inches into the soil and the number of blows counted for each 6-inch penetration interval. The numbers indicated on the logs are the total number of blows that were recorded for the second and third 6-inch intervals, or the number of blows that were required to drive the penetration depth shown; when high resistance was encountered.

Given the hammer weight and the hammer drop height used for both samplers are the same, the difference of blow counts is because of outer and inner dimensions. For the Modified California Sampler with 3 inch (76.1mm) O.D. and 2.4 inch (61mm) I.D. the reduction factor of 0.65 will be used in our project to convert NL to Ns. In Figures 48 & 49, Appendix A, variation of field SPT blows versus depth in different boreholes are shown. In these graphs, the large sampler blow counts (NL) were converted to standard SPT blow counts (NS).

The soils encountered in the borings were continuously logged in the field and visually described in accordance with the Unified Soil Classification System (ASTM D2488). The Logs of Test Borings are included in Appendix A of this report. The logs depict subsurface conditions at the approximate locations shown on the Boring Site Plans; subsurface conditions at other locations may differ from those encountered at the explored locations. Stratification lines shown on the logs represent the approximate boundaries between soil types; actual transitions may be gradual.

3.2. Laboratory Testing

The laboratory investigation was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the subsurface materials at the project site.

Selected samples retrieved from the exploratory borings were returned to the laboratory for examination and testing to evaluate their physical characteristics and engineering properties. Below is a description of the series of tests performed in our laboratory on selected samples retrieved from the field investigation. These tests were performed in accordance with the standards of the American Society for Testing and Materials (ASTM) and contemporary geotechnical engineering practices. Samples were tested to measure moisture content and unit weight, plasticity, grain size distribution, and shear strength. The results of the laboratory tests are presented in Appendix A and as appropriate adjacent to the corresponding sample designations on the boring logs.

Table 2
List of Laboratory Tests

TEST	Standard Code
Atterberg Limits	ASTM-D 4318
Grain size	ASTM-D 421 , D 422
Specific Gravity	ASTM-D 857
Water Content	ASTM-D 226
Classification	ASTM-D 2488
Direct Shear	ASTM-D 3080

4. Site Characterization

4.1. Soil Layers Description

Based on site visits and observation of retrieved samples during drilling operations, the subsurface soils consist of loose to medium dense, moist to wet, brown to grey Silty SAND overlaid on the weathered bedrock. The bedrock consists of very dense light brown weathered Sandstone with orange stains. In boreholes B-1 to B-12, bedrock was encountered at different depths with a minimum at 27 ft within B-7 and at a maximum of 46 ft in B-4. This variation is likely a result of tectonic pressures that has changed the bedrock elevation, differential weathering, and the possibility of modification by landslide mass movement.

On portions of the existing slope, man-made grading (cut and fill) has changed the soil thicknesses. In some boreholes, the soils encountered suggest Silty Sand was historically used as fill during historical grading operations that created the old reservoir at the site and/or during grading for the Lyon Tank that was constructed to replace the reservoir about 25 years ago. According to SPT blow counts, this loose to medium dense fill material is suspected to have been placed as uncompacted fill.

Based on our observations, some of the undocumented fill material is comprised of the native soils from the site making it difficult to distinguish between the two. Some boreholes were not located within the 2017 landslide area, including boreholes B-5 to B-7. The soil layers in B-1 to B-4 and B-8 to B-12 are within the landslide. The landslide mass found in these boreholes varied in thickness. The maximum 2017 landslide mass thickness observed in B-1 was 38 ft (±).

Based on the retrieved soil samples, there are areas where native soils exist above the bedrock that did not move as part of the 2017 landslide mass. These soils lie between the landslide mass and the bedrock. In Table 3, the Soil Layer Conditions after the 2017 Landslide are presented. We note, boreholes B-13 to B-16 were drilled outside of the landslide area. Therefore, the soil layer condition for these four boreholes are not described in Table 3. The landslide and native soil layers observed during drilling of the different boreholes is also presented graphically as a 3D landslide surface within the hillside. This is shown (named as Case 1) in figures 55 & 56 in Appendix B of this report.

Table 3
Soil Layers Condition After 2017 Landslide Event

Soil Layers Encountered In The Boreholes After 2017 Landslide								
BH NO.	BH depth (ft)	BH top elevation (ft)	2017 Landslide mass thickness (ft)	Thickness of undisturbed native soil below landslide layer to bedrock (ft)	Bedrock depth (ft)	Bedrock elevation (ft)	Water Surface depth (ft)	Water Surface Elevation (ft)
B-1	51.5	819.0	38	7	45	774	-	-
B-2	31.5	819.5	22	8	30	789.5	-	-
B-3	36.5	815.5	25	6	31	784.5	5	810.5
B-4	46.5	822.5	30	16	46	776.5	4	818.5
B-5	41.5	847.5	0	45	45	802.5	15	832.5
B-6	33.0	851.5	0	32	32	819.5	-	-
B-7	30.0	810.8	0	27	27	783.8	-	-
B-8	46.5	797.9	32	6	38	759.9	12	785.9
B-9	41.5	792.5	20	19	39	753.5	4	788.5
B-10	35.0	778.3	24	14	38	740.3	6	772.3
B-11	35.0	779.7	19	15	34	745.7	25	754.7
B-12	32.5	838.3	30	1	31	807.3	-	-

4.2. Groundwater

At the time of drilling, water was encountered in some boreholes at different depths. The significant difference of water level indicates that the observed water is perched water and mainly results from rainwater infiltrating at the site and at neighboring highlands and mountain slopes that then flows through permeable soils that overly the bedrock. The 2017 landslide event caused some parts of surficial soils to become scrambled and fractured, thus a change in permeability of these soil layers resulted. The wetness of the recovered interface soil samples at the slip plane contact zone indicate much higher moisture content percentage there than in the underlying weather bedrock, as a result of groundwater following the slip surface fractures. Figures 50 to 53 in Appendix A, show variation of soil saturation degree and void ratio versus depth in the different boreholes. These values were calculated using laboratory soil samples measuring dry density and moisture content.

Groundwater conditions vary with environmental variations and seasonal conditions such as frequency and magnitude of rainfall patterns. Seasonal groundwater fluctuations should be taken into account in design and construction. We recommend the contractor alert the engineers of actual groundwater levels, if encountered during construction, to determine groundwater impact on the construction procedures and on design. Inflow of groundwater during excavation could lead to significant construction problems and unsafe working conditions for personnel. If not properly controlled, groundwater inflow could also contribute to backslope failure of temporary excavations resulting in great bodily injury or death.

4.3. Soil Properties

Topographical map of the site was provided by Paul Jensen four times in February, June and October 2017, and in February 2018 to document continuing movement of the landslide mass.

The cross section locations as shown in Appendix C, were developed by HKA using the topographic map prepared by Paul Jensen. These cross sections were used as the basis for our stability analysis. The most critical cross section with deepest landslide plane was selected to carry out the slope stability evaluation.

We utilized the exploratory borings from our field investigation to develop a subsurface profile model. Four (4) different soil types were developed in these analyses.

The soil boundaries indicated on the cross sections are based on; 1) the engineer's observations and soil evaluations in the field; 2) the results of field Standard Penetration Tests (SPT) conducted during soil sampling; and 3) the engineer's laboratory test results. The soil boundary lines were projected between and beyond the location of the test borings in both directions, presuming a straight line; based on experience and engineering judgement in the site vicinity. The model is simplified and based on extrapolation of information obtained during field and laboratory testing. Changes in the soil stratum are likely more gradual than indicated in our models.

Strength parameters for the different soil types were determined using standard penetration test (SPT) results, laboratory direct shear results, and engineering judgment. The 2017 landslide was modeled using soil and bedrock parameters determined by laboratory and field test results in the way the landslide occurred and then the physical parameter accuracy was calibrated. In Table 3, in-situ landslide silty sandy layer (Soil 1), in-situ native silty sandy layer (Soil 2), and bed rock (Soil 3). The current condition of the impacted hillside was modeled using Soil 1 to Soil 3. For the improved slope condition, for those parts that were filled by compacted in-situ soil, Compact Fill (Soil 4) was introduced and used in the model.

Table 4
Soil Strengths Used For Slope Stability Analysis

Soil Type (Description, #, Model Color)	Cohesion (psf)	Friction Angle (deg)	Unit Weight (pcf)
In-situ landslide silty sandy layer # Soil 1(Yellow)	300	22	85
In-situ native silty sandy layer # Soil 2 (Light Green)	400	28	110
Bed Rock #3 (Orange)	3,000	40	125
Redensified silty sandy fill layer # Soil 4 (Light Blue)	1,500	37	115

5. Geotechnical Related Seismicity

The improvements should be designed in conformance with the most current California Building Code (2016 CBC). For seismic design, the soil properties at the site are classified as **Site Class “D”** based on definitions presented in Section 1613.3.2 in the 2016 CBC that refers to Chapter 20 of ASCE 7. The longitude and latitude were determined using a satellite image generated by Google Earth. These coordinates were taken from the approximate middle of the area of the proposed improvements:

Longitude = -121.665, Latitude = 37.127

The coordinates listed above were used as inputs in the Java Ground Motion Parameter Calculator created by the USGS to determine the ground motion associated with the maximum considered earthquake (MCE) SM and the reduced ground motion for design SD. The results are as follows:

<u>Site Class D</u>

SM_S= 1.500 g

SM_1= 0.902 g

SD_S= 1.000 g

SD_1= 0.601 g

A maximum considered earthquake geometric mean (MCE_G) peak ground acceleration (PGA) was estimated using the Figure 22-7 of the ASCE Standard 7-10. The mapped PGA was 0.527 g and the site coefficient F_{PGA} for Site Class D is 1.0. The MCE_G peak ground acceleration adjusted for Site Class effects is PGA_M = F_{PGA} * PGA

PGA_M = 1.0 * 0.527g = 0.527 g

6. Quantitative Slope Stability Analysis

Stability analysis was performed on a cross section cut through the project site. The selected cross section location was selected by HKA's Project Geologist. The slope stability analysis was performed to quantify the instability associated with the occurrence of the 2017 landslide using the 2017 slope geometry; and also to analyze the potential for failure of the proposed improved slope under static winter conditions and seismic loading conditions.

6.1. General Methodology

Slope failures or landslides can cause problems including encroachment, property damage, personal injury, or even death. Failures of slopes occur when stress acting on the soil mass is greater than its internal strength (shear strength). A slope is considered stable when the strength of its soil mass is greater than the stress field acting within it. Some common variables influencing stress are gravity (steeper slopes), hydrostatic pressure (perched groundwater), soil surcharge pressures (overburden), concentrated surcharge at up slope (buildings, vehicles on the road and etc…) and seismic surcharge (earthquake shaking).

Various methods of analyzing stability of slopes yield a factor of safety. A factor of safety (FS) is determined by dividing the resisting forces within the slope soils (earth materials) by the driving forces within the slope (stress field). A FS greater than or equal to 1.0 is considered to be in equilibrium. A FS less than 1.0 is a potentially un-stable slope condition. HKA considers the potential for instability of a slope or hillside to be low with a FS against sliding greater than or equal to 1.10 under seismic loading conditions and 1.50 under static loading conditions. Some governing agencies including Santa Cruz County Environmental Planning and the Mining Safety and Health Administration (MSHA) require slopes to have a FS equal to or greater than 1.20 to be considered seismically stable.

6.2. Quantitative Analysis with GEO-SLOPE 2018

The analysis was completed with the aid of GEO-SLOPE computer software version 9.0.3.15488. A model for the section was defined with the input parameters consisting of slope geometry, soil properties, loading conditions, and pore water pressure ratio. Each model was evaluated under static and seismic loading. Mohr-Coulomb material model was used to define the soil properties. The analysis calculates the factor of safety against sliding for the failure surface(s).

Trial failure surfaces for the analyses consisted of circular (general) and wedge type failures. Morgenstern-Price analysis method is used to determine normal and resistive forces in each slice. The forces in each slice are then summed up for total force acting on the mass. In circular (general) failure mode stability assessment, the computer program assumes many failure surfaces using initiation and termination points on the ground surface selected by the user. These chosen points represent the toe and scarp of each potential landslide in relation to the assumed failure surfaces. The critical trial failure surface from the pseudo static analysis condition was selected as the projected failure surface in the development of design parameters.

6.3. Seismic Coefficient

The ground motion parameter used in pseudo static analysis is referred to as the seismic coefficient “k_h”. The selection of a seismic coefficient has relied heavily on engineering judgment and professional publications. The 2016 California Building

Code (CBC) provides site class definitions for seismic design of structures. Based on these definitions, a review of the site soil properties presented on our soil boring logs, the site is classified Site Class D, in accordance with ASCE 7 (with March 2013 errata). The current version of the California Building Code contains reference to maps of peak ground acceleration (PGA) based on site latitude and longitude. For this project the mapped PGA is 0.527g. The PGA is multiplied by a factor related to the seismicity of the site to obtain the seismic coefficient.

Two empirical charts developed by Blake and others are currently available for estimating the seismicity factor in Figure 1 and Figure 2 of Chapter 5 Analysis of Earthquake-Induced Landslide Hazards in CGS *Special Publication 117 Guidelines For Analyzing and Mitigating Seismic Hazards in California 2008.* Each chart represents a minimum allowable displacement of the embankment or slope. Figure 1 is a minimum allowable displacement of 2 inches and Figure 2 is a minimum allowable displacement of 6 inches. In general, the more displacement the slope can tolerate, the lower the seismicity factor or percentage of PGA can be calculated. A simple way to think of it is if a maximum of 0 inches of displacement is tolerable then k_h =100% of PGA would be calculated. If the slope can tolerate a maximum of 6 inches of movement then k_h would be much closer to 50% of PGA. If the k_h value used results in a factor of safety less than 1.2 for seismic loading conditions and 1.5 for static, a Newmark analysis should be completed.

For this analysis, a maximum displacement of 2 inches within the failure mass was presumed to be tolerable. This presumption is typical for stability analysis involving structures or permanent improvements. The seismicity factor was estimated to be 54.0% of PGA or $k_h = 0.54 * 0.527g = 0.285g$.

6.4. Geometric Assumptions

Six (6) geometric sections (A3, B3, C3, D3, E3 & F3) were prepared by HKA's Geologist using the topographic map and ground surface profiles prepared by the Surveyor. For our analysis, the failure surface was focused along the worst case cross section (C3) which has the deepest impacted layers in 2017 landslide event. Four (4) soil types are presented in our model. At the ground surface of the slope either in-situ land slide silty sandy layer or in-situ compacted silty sand fill (for improved slope condition) exist. Beneath the landslide layer, the native soil consists of silty sand. Below the native soil, very dense bedrock is encountered. A phreatic water surface was observed in some boreholes at different depths; and in some boreholes, no water surface was encountered. The landslide happened after an above average rainy season, and landslide movement resulted in ground fractures that act as groundwater conduits. Therefore, the top soil has the potential to become partially saturated. In order to consider the effect of rainfall in creating pore water pressure, an "Ru" coefficient is considered for the in-situ landslide material and in-situ compacted fill. Ru simply models the pore pressure as a fraction of the vertical earth pressure for each slice. Each soil can have a different Ru value. In our project model, the Ru for the top soil (either in-situ compact fill or in-situ land slide sand) was designated as 0.4.

6.5. Slope Stability Models for Studied Site

The project slope has been modeled in four (4) conditions and each model has been evaluated in both static and seismic conditions. The models are introduced as follows:

a. 2017 Landslide event re-creation; Based on engineer's judgment of the landslide geometry.
b. Current condition of the existing slope after landslide;
c. Improved slope installing two rows of secant piles, one along the outboard side of the lower road and another on the hillside mid-way to the upper road. The model also considers removal of 5 (+/-) feet from the surface of the landslide from the upper road to just downslope from the lower road. After the soil is removed and off hauled, an additional 5 (+/-) feet below this is also removed and redensified as engineered fill. An engineered fill slope with a gradient of 2H:1V is constructed with its toe starting at the upper row of secant piles and crest along outboard side (shoulder) of the upper road. The engineered fill extends below the upper road restoring access.
d. The landslide mass below the lower row of secant pile with and without a surcharge load. The surcharge is the soil removed from the landslide between the upper and lower roads.

The slope stability safety factor for the above models in both static and seismic conditions are shown in Appendix C graphically and tabulated in this section of the report..

a. 2017 Landslide Event Recreation

Distinguishing the landslide mass layer from native soil is one of the most important goals of the project investigation. In some areas the landslide mass and native soil layers are the same but some native soil was below where the 2017 landslide slide plane formed. If desired, further exploration involving large diameter exploratory borings would be required to absolutely define the landslide mass thickness throughout the landslide area. In order to approximate the thickness of the landslide mass and native soil thicknesses, and also the depth to bedrock in each borehole, laboratory and field test results have been considered. Borehole logs and the soil samples retrieved from drilling were observed as well. Then a model of 2017 slope was estimated. In order to select the soil parameters most accurately, the soil parameters have been adjusted in a way that the 2017 landslide mass thickness depicted in the model matches the 2017 landslide layer thickness that was defined by HKA. The achieved (adjusted) soil layers' parameters were used in the other models.

b. Current Condition Slope Stability Evaluation

In order to determine the future stability of the existing slope which contains the existing active landslide layers, the current condition of the slope has been modeled using the cross section C3 provided by Project Surveyor. Stability safety factors under static and seismic conditions have been evaluated. One of the most important results of current condition slope modeling is to evaluate the behavior of the native soil overlaid on the bedrock and to understand if the native soil will participate in future landsliding under design conditions and if the answer is yes, then how deep will be the future slope failure plane be?

Figures 63 & 64 in Appendix C show the safety factor of the current condition slope stability. The result shows that in a probable predicted future earthquake event, the existing slope can not be stable and will likely fail. The existing landslide mass will continue to be unstable under static condition as well. Under a seismic condition, all of the soil layers above the bedrock will likely be involved in the slope failure. In order to highlight the present failure condition, shading contours have been provided. As discussed previously, the slope is considered relatively stable if the safety factor under a seismic condition is more than 1.1. As can be seen in Figure 64 in Appendix C, the dark blue shading contours show a safety factor equal to or greater than 1.1 and the border of the light and dark blue zones shows the depth of future probable landslide planes and the thickness of the resulting probable future landslide mass. This border mostly touches the bedrock which indicates the predicted potential future landslide includes the native soil layers below the 2017 landslide mass and thus are susceptible to future landsliding. Therefore, slope stabilization should be considered at least as deep as the top surface of the bedrock.

Discussion about Slope Stability Improvement Options

Several alternative methods to improve the existing slope were assessed. As discussed earlier, in a future probable earthquake event, deep landsliding is expected. The in situ native soil layers above the bedrock will become part of the landslide. The bedrock was encountered in B-1 and B-2 at 45 feet and 30 feet respectively. Because the depth of the probable landslide is significant, some of the alternative methods are likely not practical or make the stabilization very costly. HKA previously submitted a memorandum letter on September 15th, 2017 that discussed several slide repair options and their feasibilities from a geological and construction perspective. These options are presented briefly as follow:

- ***Remove and Replace The Entire Slide Mass as Engineered Fill*;** This method is not practicable because the existing saturated landslide mass materials are not qualified in their in-situ condition for use as engineered fill; and there is little to no room onsite for material conditioning (moisture conditioning or drying back as needed) or hauling the removed the soil offsite for storage and conditioning.

- ***Dewater Slide Mass and Stabilize Road*;** This is not considered feasible because it is difficult to locate and isolate the source of subsurface water; Moreover, the existing slope is not stable seismically even under dry soil conditions.

- ***Tieback Soil Pin Pile Walls Below Both the Upper and Lower Roadways*;** This option is likely to be very costly and difficult to construct. Tiebacks will be very long in order to fully penetrate the landslide zone and extend a sufficient length into the stable bedrock zone to provide stabilization. Drilling long inclined tieback holes is difficult. They may need casing to prevent the hole wall from collapsing where it is within the landslide mass. Landslide soil layers can not provide arching stability and will collapse between the pin piles. The wall would need to be installed very deep and seated on the bedrock. Access roads to support drilling equipment would need to be constructed.

- ***Install culvert in stream and excavate upper slide mass; place and compact excavated spoil over pipe; construct retaining wall to stabilize upper roadway*;** This option is feasible and physically practical, but may not be permitted by regulatory agencies if another option is deemed less environmentally damaging.

The best solution for deep landslide stabilization may be a combination of feasible methods such as improving the drainage system for the site, excavating and removing the upslope area of the landslide mass soils then placing the excavated soil at the lower parts of the slope over a new culvert placed in the streambed, then recompacting that soil to achieve a compacted fill that sufficiently buttresses the slope to make reconstructed segments of the upper and lower roads stable. This option is likely to also require installation of vertical reinforcement such as piles into the slope. The permit process for this option may prove be very difficult with many agencies involved.

This option of repair must be modified as appropriate depending on the type of vertical reinforcement and its installation location. The upper road is located within the landslide scarp area. The slope crown should be stabilized where there are critical structures above it that must be protected from sliding. The large commercial water tank in its current location is setback beyond the influence of the subject landslide. If the proposed fill slope grading and flattening of the 2017 landslide scarp can provide a stable service platform for the upper road, then it is not necessary to install vertical reinforcement or tiebacks at the upper part of the slope. Construction of a wall between the pin piles is not feasible because the wall would have to be deep, at least reaching the bedrock. Also, it is impractical to remove the existing landslide soil mass and install the wall. If the wall is a sheet pile wall driven into the soil, specialty equipment must gain access to the work area and thus roadbed improvement would need to be provided.

c. ***Improved slope; installing two rows of secant piles one along the outboard side of the reconstructed lower road and the other on the hillside mid-way to the upper road. Constructing engineered fill slope***

from upper row of secant piles to inboard side of reconstructed upper road restoring access and road shoulder.

If the goal is to stabilize the existing slope containing the landslide mass, two rows of secant piles should be installed which extend to a depth with at least a minimum 15 feet embedment into the bedrock. The landslide soil hasn't enough strength to stand between pin piles or widely spaced piers based on principles of arching. Therefore, zero spacing between piles is a requirement. Secant piles in this case are vertical piling that are installed next to each other with no space between each adjacent piles. The secant piles wall is constructed using a series of closely spaced drilled shafts filled with reinforced concrete. The piles can also be driven. However, driving the piles into very dense bedrock can be challenging or impractical. Also, if cast-in-place piles are designed for the project, boreholes in the landslide mass are expected to need casing to prevent their sidewall soils from collapsing into the drilled borehole.

Based on the slope stability results, each secant pile should have minimum 20,000 pounds per foot lateral capacity. For preliminary design purposes it can be assumed the resultant force acts at a location 2/3 the depth to the bedrock within the overburden soil. For practical applications, the actual location of the resultant should be determined by HKA at several sections along wall profile. In Figures 67 & 68 in Appendix C, slope stability safety factors under both static and seismic conditions are presented. Based on the slope stability evaluation results, the safety factors for both static and seismic conditions are greater than the minimum acceptable limits, which means the uphill side of the slope starting from the outboard side of the lower road will be stabilized after installing two rows of secant piles and constructing an engineered fill slope to support the upper road. However, the proposed secant piles do not provide stability against sliding for soil on the downslope side of the lower row of secant piles. It is assumed the soil down slope from the secant piles (including the portion of the existing 2017 active landslide mass located there) will continue to move or eventually mobilize into the creek at some point in the future.

For the project slope stability analysis, the most critical cross section with deepest landslide material has been considered which requires installation of long and deep secant piles. The length of the piles will be reduced when moving toward the flanks (sides) of the landslide mass. In order to get a better understanding, a 3D view of the slope and potential landslide layers in the boreholes (case 2) are presented in Figures 57 to 58 in Appendix B.

The top 5 (+/-) feet of soils from just upslope from the upper row of secant piles to approximately 40 feet below the lower secant pile row is proposed for removal. These soils should be temporarily stockpiled in an approved location by HKA for re-use. An additional 5 (+/-) feet of soil below the removed soils will also be removed (10 feet total) and redensified back into place. The stockpiled soils, if dried back or moisture conditioned as needed, may be re-used as engineered fill during construction of the fill slope that will support the upper road. The remaining soil left over should be off hauled or placed as engineered fill in a location on-site approved by HKA. The material should not be placed on the landslide mass below the lower row of secant piles as it can exacerbate instability of this soil mass.

By using this method two positive things with respect to landslide resistance are accomplished. The first is the existing bulging landslide soil mass will be removed from the upslope area effectively un-loading this portion of the hillside and reducing the driving force acting on the landslide mass. The second is, if the removed landslide mass can be dried back to near optimum moisture content it can be re-used as engineered fill during construction of the fill slope that will restore the travel way and shoulder of the upper road.

The graphical stability models presented in Figures 69 & 70 of Appendix C present general failure surface below the engineered fill slope that will support the upper road. The analysis of these models considers the stability of the fill slope without the benefit of the influence from the upper row of secant piles. The general failure surfaces were generated to slice through the landslide soils below the engineered fill slope. Our calculated factors of safety against sliding under static wet winter and seismic shaking conditions are above the minimum acceptable values for modern geotechnical engineering practice.

d. Stability of the landslide mass below the lower row of secant piles after installation with and without a surcharge load.

The stability of the landslide mass below the lower road after the secant piles have been installed was quantified with and without a surcharge load. The purpose of these models was to demonstrate the negligible effect of the secant piles on the stability of this portion of the landslide mass. The results of the analysis indicate that this portion of the landslide mass is unstable in static and seismic conditions. The factors of safety against sliding are less than 1.0 for both loading cases. When the surcharge load of soil was placed upon this portion of the landslide it further reduced the factors of safety against sliding. This indicates that placement of soil upon the lower landslide mass may exacerbate instability or speed up movement of the soil mass into the creek and is not recommended. This model was evaluated to quantify the effects of placing soil removed from the upper portion of the landslide onto to the lower portion of the landslide as a means of disposal. This is presented graphically in Figures 71-74 in Appendix C of this report.

6.6. Slope Stability Conclusions

The slope stability assessment is for general (global type) slope failure and consists of initiation and termination of trial failure surfaces on the top and toe of slopes for recreation of landslide and evaluation of existing condition. The models with slope improvements including secant piles and engineered fill were evaluated with failure surfaces running top to toe as well as mid slope as selected by engineer to evaluate benefit to stability of improvement. In both scenarios, the trial failure surface passes through the soil layers in the cross section model. The general shear trial failure surface screens for potential instability below the in-situ landslide and native soil layer. The in-situ landslide soil layers were also screened for trial failure surfaces localized within the soil layer.

In table 5, slope stability analysis results for the four (4) aforementioned models static and seismic conditions are shown.

In summary the large landslide soil mass can be stabilized from the lower row of secant piles along the outboard side of the lower road up to the inboard side of the re-constructed upper road. For stability discussion purposes we will refer to this as the “upper landslide” and the portion downslope from the lower row of secant piles the “lower landslide”. A second row or upper row of secant piles on the hillside mid-way to the upper road is required to stabilize the upper landslide soil mass described in this conclusion. Factors of safety against sliding are greater than what is considered stable using modern geotechnical engineering standards.

Although the secant piles will restore stability of the upper landslide it will not restore access across the upper road. An engineered fill slope is modeled to support and restore the upper road. The fill slope is modeled to have a 2H:1:V slope gradient with its toe at the upper row of secant piles and crest at the shoulder of the upper road. The fill slope would extend to allow reconstruction of the upper road to allow vehicular traffic, and would terminate along the inboard cut slope of the upper road.

HKA re-iterates that the stability models indicate that the presence of improvements such as secant piles and engineered fill will stabilize the upper area of the landslide, but will not stabilize the lower area of the landslide. The removed soil during grading can be re-used as engineered fill in construction of the fill slope restoring the upper road. Any excess soil should not be disposed of by placing it upon the lower landslide. Doing so may exacerbate instability by increasing the rate of mobilization of the lower slide into the creek below. If SLVWD would like to stabilize the lower landslide area and dispose of soil in this location, additional rows of secant piles will need to be constructed within the lower landslide along with retaining walls to buttress the fill along the toe. The location and depth of these improvements should be carefully evaluated by HKA.

We anticipate a temporary road will need to be constructed to install the upper row of secant piles on the hillside. Additional working meetings with structural and civil designers, specialty contractors, and SLVWD are anticipated to develop viable working drawings.

6.7. Limitations of Analysis

It must be cautioned that slope stability analysis is an inexact science; and that the mathematical models of the slopes and soils contain many simplifying assumptions, not the least of which is homogeneity. Density, moisture content and shear strength may vary within a soil type. There may be localized areas of low strength or perched ground water within a soil. Slope stability analyses and the generated factors of safety should be used as indicating trend lines. A slope with a safety factor less than one will not necessarily fail, but the probability of slope movement will be greater than a slope with a higher safety factor. Conversely, a slope with a safety factor greater than one may fail, but the probability of stability is higher than a slope with a lower safety factor.

Table 5
Slope Stability Analysis Results

Condition	Loading Condition	Minimum Factor of Safety Against Sliding	Trial failure Surface Shape
2017 Landslide Event	Static	0.92	Circular
2017 Landslide Event	Seismic	0.45	Circular
Current Condition	Static	0.96	Circular
Current Condition	Seismic	0.48	Circular
Improved Slope by Installing Two Rows of Secant Piles With Shallow Redensification of Upper Landslide	Static	3.54	Circular
Improved Slope by Installing Two Rows of Secant Piles With Shallow Redensification of Upper Landslide	Seismic	1.19	Circular
Upper Road Slope Stability General Failure Safety Factor after proposed upslope grading and head scarp flattening	Static	2.73	Circular
Upper Road Slope Stability General Failure Safety Factor after proposed upslope grading and head scarp flattening	Seismic	1.49	Circular

General Failure Safety Factor of Landslide Mass Below Lower Secant Row	Static	0.96	Circular
General Failure Safety Factor of Landslide Mass Below Lower Secant Row	Seismic	0.54	Circular
General Failure Safety Factor of Landslide Mass Below Lower Secant Row With Surcharge	Static	0.52	Circular
General Failure Safety Factor of Landslide Mass Below Lower Secant Row With Surcharge	Seismic	0.91	Circular

7. Building Codes and Site Class

Project design and construction should conform to the following current building codes:

-2016 California Building Code (CBC); and

-2016 Green Building Standards Code (CAL Green)

In accordance with section 1613.3.2 of the 2016 CBC, the project site should be assigned the Site Class D.

8. Recommendations for Design and Construction

The results of our investigation indicate that the different slope improvement / stabilization options are feasible from a geotechnical standpoint. The criteria and recommendations presented in this report are focused on the secant pile repair schemes previously presented in the report and we recommend that those should be followed during design and construction of the project.

Geotechnical considerations at the referenced site include improving the stability of the upper and lower roads crossing the existing landslide, the potential for strong seismic shaking, and providing adequate site drainage provisions.

Our slope stability analysis results have shown that the current condition of the existing overburden soils overlying the bedrock (including both landslide mass and native soil materials) have high instability potential when moistened or saturated during heavy rainfall or during the occurrence of an earthquake. The instability is possible under both static and seismic conditions. Our basis of design is reliant on the potential slip planes derived from the slope stability analysis. The failure planes were considered to toe out on the slope, based on our best estimate of the soil/bedrock contact and also were controlled by the position of the creek as the base of the slide. The geotechnical considerations for the failure condition are related to the geometry of the slope and soil information determined from the test borings, Figures 5 through 26 in Appendix A.

To mitigate the instability potential, it is recommended to unload the upper landslide by removing the upper 5 (+/-) feet of soil. After removal of the soil an additional 5 (+/-) feet of the upper landslide should be also removed, but this soil re-densified back into place as engineer fill. The upper landslide should be stabilized using two rows of buried secant piles. The lower row of secant piles would be constructed along the outboard side of the lower road and is estimated to be 200 feet long by as much as 55 feet deep. The upper row of secant piles is recommended to be constructed on the hillside mid-way to the upper road. The upper row is estimated to be 150 feet long by as much as 40 feet deep. The piles should be advanced a minimum 15 feet deep into the bedrock.

To rebuild and secure the severely damaged portion of the access road where the landslide mass dislodged from the head scarp, an engineered fill slope is recommended to be constructed from the upper row of secant piles up to the inboard side of the upper road.

Due to disturbance of the soil during the 2017 landslide event, the existing landslide soil layers have residual strength which are significantly less than the strength of the native soil. Therefore, it is expected that the landslide soils cannot provide arching. So, there should be no room between two adjacent consecutive piles along the respective wall alignments. Based on the slope stability results, the safety factor for the slope stability both in static and seismic conditions are greater than the minimum acceptable limit. The stability model indicates that if two rows of secant piles are inserted into the model, the unsupported part of the slope located downslope from the piles is considered unstable. In reality, and in the long term, that part of the unsupported slope and landslide mass will likely continue to slide toward the creek at the northern end of the property. Disposal of soil or any other surcharge load placed onto the lower landslide mass will exacerbate instability and is not recommended.

In the aforementioned slope improvement method, we recommend excavating the surficial soils on the slope in the upslope area of the landslide mass. Some portions of the excavated soil will need to be moisture conditioned or dried back as needed, replaced and recompacted at the initial location to remove the existing bulge in the landslide mass that exists below the landslide headscarp (formed during the 2017 landslide event) to make a uniform firm surface and to provide a flatter slope. The rest of the excavated soil may be re-used in construction of the engineered fill slope that will restore the travel way and shoulder of the upper road. Excess soil not used as described above may be placed as engineered fill in other locations on the property approved by HKA. The excess soil should not be disposed of upon the lower landslide mass.

An advanced widespread drainage system should be considered for the project site to collect the runoff water from the hillside. A proper site drainage system is important for the long term performance of the site. As indicated elsewhere in this report, perched water was observed in some of the drilled boreholes. Though groundwater levels could not be studied for this site, the reported observations indicate groundwater collects within the in-situ soil, thus, the proposed slope improvement should include subdrains as part of the site's planned remediation. To minimize the impact of subsurface seepage on the improved slope, subdrains are recommended.

HKA would like to have working meetings with client's representative and project designers when the slope improving option enters a conceptual design phase to discuss more about the limitations of our model. The variable depth of the landslide from its deepest point along the center to the flanks where it pinches out to nothing should be carefully considered. The varying depth of the slide will have great affect on the location and magnitude of the resultant force. HKA should work with the civil and structural designers to develop additional models in select locations to optimize a value engineering type of solution. To accomplish this additional testing may be needed such as down hole borings and or geo-physical survey to fine tune the 3-D model of the landslide soil mass. Soil pile interaction using a finite method can also aid in value engineering design.

The following recommendations should be used as guidelines for preparing project plans and specifications:

Site Grading (Fill/Cut Slopes)

1. The HKA should be notified ***at least four (4) working days*** prior to any site clearing or grading operation so that the work in the field can be coordinated with the Grading Contractor and arrangements for testing and observation services can be made. The recommendations of this report are based on the assumption that the HKA will perform the required testing and observation services during grading and construction. It is the client's

responsibility to make the necessary arrangements for these required services.

2. Where referenced in this report, Percent Relative Compaction and Optimum Moisture Content shall be based on ASTM Test Designation D1557-latest revision.

3. Areas to be graded should be cleared of obstructions including loose fill, or other unsuitable material. Existing depressions or voids created during site clearing should be backfilled with engineered fill.

4. Cleared areas should then be stripped of organic-laden topsoil. Stripping depth should be from 2 to 4 inches. Actual depth of stripping should be determined in the field by an HKA representative. Stripping should be wasted off-site or stockpiled for use in landscaped areas if desired.

5. Areas to receive non-expansive engineered fill should be scarified 8 inches, moisture conditioned to over optimum moisture content, and redensified to 90 percent of maximum density. Portions of the site may need to be moisture conditioned or dried back as needed to achieve suitable moisture content for compaction. These areas may then be brought to design grade with engineered fill.

6. Engineered fill should be placed in thin lifts not exceeding 8 inches in loose thickness; moisture conditioned, and compacted to at least 90 percent relative compaction.

7. We understand grading at the site will consist of excavation of a portion of landslide overburden soil to construct a flatter slope along the upper landslide to allow for installation of the upper row of secant piles. A temporary access road and working platform will also need to be constructed to support heavy equipment that will be required to advance the secant piles.

8. The top 5 (+/-) feet of soils from just upslope from the upper row of secant piles to approximately 40 feet below the lower secant pile row is proposed for removal. These soils should be temporarily stockpiled in an approved location by HKA for re-use. An additional 5 (+/-) feet of soil below the removed soils will also be removed (10 feet total) and redensified back into place. The stockpiled soils, if dried back or moisture conditioned as needed, may be re-used as engineered fill during construction of the fill slope that will support the upper road. The remaining soil left over should be off hauled or placed as engineered fill in a location on-site approved by HKA. The material should not be placed on the landslide mass below the lower row of secant piles as it can exacerbate instability of this soil mass.

9.¶ To rebuild and secure the travel way and shoulder of the upper access road where the soil mass dislodged from the head scarp an engineered fill slope with a gradient of 2H:1V is recommended to be constructed from the upper row of secant piles up to the inboard side of the damaged upper road.

10. Areas to be graded should be cleared of all obstructions, including foundations and structures if exist, old fill, trees not designated to remain and other unsuitable material. Disturbed soil resulting from demolition and clearing operations may be stockpiled for use as engineered fill, provided the fill is clean of organic material, unacceptable colluvium deposits or other unsuitable material. Existing depressions or voids created during site clearing should be backfilled with engineered fill.

11. If project site grading is performed during or shortly after the rainy season, the grading contractor may encounter compaction difficulty, such as pumping or bringing free water to the surface. If compaction cannot be achieved after adjusting the soil moisture content, it may be necessary to over-excavate the subgrade soil and replace it with angular crushed rock to stabilize the subgrade. We estimate that the depth of over-excavation would be approximately 12 inches under these adverse conditions.

12. Import soils if utilized as engineered fill at the project site should:

1) Be free of wood, organic debris and other deleterious materials;

2) Not contain rocks or clods greater than 5 inches in any dimension;

3) Not contain more than 25 percent of fines passing the #200 sieve;

4) Have a Sand Equivalent greater than 18;

5) Have a Plasticity Index less than 18;

6) Have an R-Value of not less than 30; and

7) Contractor should submit to HKA samples of import material or utility trench backfill for compliance testing a minimum of 4 days before it is delivered.

13. We estimate shrinkage factors of 15 to 25 percent for the on-site materials when used in engineered fills.

14. Cut and fill slopes should be protected from erosion by preventing runoff from spilling over graded slopes. Generally, Lined V-ditch and/or curtain drain at the top of the hillside and curtain drain at the secant piles wall may be considered for long-term drainage control. A proper drainage system should be designed for the entire site to collect and control the run off waters.

15. After the earthwork operations have been completed and HKA has finished observation of the work, no further earthwork operations shall be performed except with the approval of and under the observation of HKA.

16. Permanent graded slopes should be constructed no steeper than 2.H:1V (horizontal to vertical). Graded slopes are expected to require erosion control and periodic maintenance for surface sloughing.

17. Fill slopes should be constructed with keyways and benches sloped in the inboard direction a minimum 5 percent. The keyways should be a minimum 8 feet wide and placed over bridging material comprised of 12 inches of gabion over geogrid equivalent to Mirifi 600X of better. The keyway and benches should be constructed with drains to alleviate hydrostatic pressure. The geotechnical engineer should approve the type of drainage system and location for discharge.

Secant Pile Walls

18 Secant pile walls are formed by constructing intersecting reinforced concrete piles. Secant pile walls are formed by keeping spacing of piles less than one diameter. Secant pile walls are used to build cut off walls for the control of ground water inflow and to minimize movement in weak and wet soils.

19. Secant walls are constructed in the form of hard/soft (or firm) or hard/hard walls on adjacent piles. If the distance between the hard and soft piles are equal to piles diameter, the wall is called tangent pile wall.

20. The secant piles are reinforced with either steel rebar or with steel beams and are constructed by either drilling under mud or augering or driving. In this wall system, there are two types of piles. Primary piles are installed first. These piles are mainly responsible for waterproofing and filling the voids.

21. In the Hard/Firm (or soft) wall system, the primary piles have no reinforcement and consists of flexible concrete that can be cut while the secondary piles are installed. The secondary piles which should have reinforcement will be installed between the primary piles once the latter gain sufficient strength. Where short term water retention is required, this system offers the most cost-effective and rapid solution. The wall consists of interlocking bored or driven piles. Primary piles are constructed first using a ‘soft’ cement-bentonite mix or ‘firm’ concrete. Secondary piles, formed in structural reinforced concrete, are then installed between the primary piles. The primary piles in Hard/Firm (or Soft) wall system should be drilled to minimum bed rock depth and the pile base will be sited on the bedrock. Therefore, all the lateral capacity of the wall will be provided by the secondary piles and therefore, the secondary piles design in hard/firm (or soft) wall system differs from the hard/hard wall system design.

22. Hard/hard wall construction is very similar to a hard/firm wall but in this case the primary piles are constructed in higher strength concrete and may be reinforced. Heavy duty rotary piling rigs, using tools fitted with specially

designed cutting heads, are necessary to cut the secondary piles. The end product provides a fully concreted face and can be an effective alternative to diaphragm wall construction.

23. Pile overlap is typically in the order of 3 inches. In a tangent pile wall, there is no pile overlap as the piles are constructed flush to each other.

24. Verticality tolerances may be hard to achieve for deep piles. Special care should be taken to assure the pile installation is vertical.

25. Special construction method might be required to make sure that total waterproofing is provided.

26. A monitoring and maintenance program is an integral component of the design of secant pile wall. To maintain the integrity of the wall system, it is necessary to conduct regular inspections of the slope and the secant pile. We recommend secant pile wall be inspected after long duration winter storms, severe seismic shaking, and at least once every 2 years by a licensed engineer or an engineering geologist to monitor the status of the wall system and recommend maintenance when needed.

Drilled piles for the secant Pile Wall

27. If cast-in-place piles are considered for secant pile wall system, the project

site secant piles should be excavated prior to placement of the reinforcement cage. All pile excavations should be observed by the soils engineer prior to placement of steel and concrete. Pile diameter is to be determined by the project structural engineer. Pile drilling sequence and method of pile drilling is to be determined by the project contractor. Casing of the pier shaft within the loose sandy soils may be required.

28. Secant piles at the project site should be embedded a minimum of 15 feet into the competent bedrock.

29. The landslide layers over the native soil are considered residual strength disturbed soil. The behavior of this layer is not uniform at different locations and depths of the slope. Therefore, it is prudent to neglect the top of the secant piles for calculating passive resistance. This length is decreased as they reach the flanks (sides) of the slope which contain shallower landslide deposits. At present, the only reliable information of landslide thicknesses is the existing geotechnical boreholes. Therefore, complementary investigation to determine the exact thickness of the landslide layer at the different locations of the slope should be performed or conservative landslide depth should be assumed for designing.

30. The secant piles are installed next to each other without any room for soil to provide arching. Therefore, if applicable for pile designing, arching capability

factor and safety factor should be considered equal to 1.0.

31. At 15 feet below bedrock, an allowable vertical bearing and tension capacity pile of 15 ksf and 6 ksf respectively plus a one third increase for short duration loading may be used for design of the drilled piers. It must be noted that side friction for soil layers overlaid the bedrock has been disregarded due to existing residual soil.

32. Total and differential settlement for the secant piles penetrating the looser landslide and native soil deposits to be embedded within the bedrock, are anticipated to be less than 1 inch and 0.5 inch respectively.

33. Prior to placing reinforcing steel and concrete, all pile excavations should be thoroughly cleaned. The foundation excavations must be observed by HKA prior to placing reinforcing steel and concrete.

34. The Contractors are responsible for following CAL-OSHA regulations, local codes and ordinances and any requirements outlined on any project plan sheets to maintain a safe working environment at the project site.

Active and Passive Pressures

35. The active pressures, as an equivalent fluid pressure, for both undrained and drained conditions under static and seismic conditions are presented in Table 6.

Table 6
Recommended active pressures

Recommended Active Pressure EFW (pcf)	Landslide Soil	Native Soil	Bed Rock
Undrained / Static condition	75	79	76
Undrained / Seismic condition	77	83	82
Drained / Static condition	42	39	28
Drained / Seismic condition	48	47	40

36. The passive pressure available in the soils below the bottom of the excavation may are presented as an equivalent fluid pressure:

Table 7
Recommended passive pressures

Recommended Passive Pressure EFW (pcf)	Landslide Soil	Native Soil	Bed Rock
Undrained / Static condition	120	200	350
Undrained / Seismic condition	100	175	300
Drained / Static condition	190	310	550
Drained / Seismic condition	125	250	480

37. Aforementioned drained condition earth pressure values are assumed when walls are fully drained to prevent hydrostatic pressure behind the walls. Drainage materials behind the wall should consist of Class 1, Type A permeable material complying with Section 68 of Caltrans Standard Specifications, latest edition.

<u>Driven Piles</u>

38. Vertical alignment of the piles should be preserved during driving. However, an inclination of 2 to 3 inches from vertical can be accepted as the tolerance for such piles.

39. In a group of piles, the middle piles should be driven first and then working towards the perimeter piles. This is to prevent displacement of the already driven piles due to the lateral movement of the soil. In the granular soil if the piles are driven at spacing of less than three times the diameter of the adjacent pile, due to densification of the soil, penetration would be difficult.

40. When excessive resistance to the driving is mobilized, the operation can be stopped. If the pile is penetrated less than the calculated depth, the operation can be halted for one week in order to dissipate the excess pore pressure generated in the soil. The driving should be resumed after this period. However, if still the required penetration is not achieved, a pile load test is proposed to check the capacity of the driven pile.

41. If pile heave is observed, level readings referenced to a fixed datum shall be taken by the Engineer on all piles immediately after installation and periodically thereafter as adjacent piles are driven to determine the pile heave range.

42. If during the driving process for adjacent piles, piles shall be re-driven:

- For end bearing piles, if the heave is more than 0.5 inch.
- For shaft friction piles, if the heave is more than 1.5 inch.

Surface & Subsurface Drainage

43. The surface drainage from within the slipout area needs to be collected and directed to catch basins, existing creek or to outside of the site. Most importantly surface drainage should not be allowed to runoff or spill over the edge of the fill. The collected runoff should be piped down past the secant piles wall and downslope as well. Subsurface drains should be installed at the contact of recompacted topsoil on the slope. The number of drains and spacing should be determined by the project Civil Engineer. The drains should collect subsurface drainage within the improved area and convey drainage to an adequate discharge point downslope of the improvements.

44. Surface drainage should include provisions for positive gradients so that surface runoff is not permitted to pond adjacent to pavements nor spill over the slope. Surface drainage should be directed away from the graded slope.

45. The migration of water or spread of extensive root systems below excavations, embankments, foundations, slabs, or pavements may cause undesirable differential movements and subsequent damage to these structures. Landscaping should be planned accordingly.

Monitoring

46. A survey-monitoring program should be implemented to monitor slope

displacements during construction. In addition, improvements should also be surveyed and photographs and/or video taken to document baseline conditions. The deflection at the top of the secant piles should be surveyed periodically. If the piles head deflect significantly or if distress or settlement is noted adjacent to the top of the piles, an evaluation should be performed and corrective measures taken.

Plan Review, Construction Observation, and Testing

47. Haro, Kasunich and Associates should be provided an opportunity to review project plans, prior to construction, to evaluate if our recommendations have been properly interpreted and implemented in the design. Having done so, we can prepare the county-required geotechnical plan review letter.

48. If we do not review the plans and provide observation services during the earthwork phase of the project, we assume no responsibility for misinterpretation of our recommendations.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

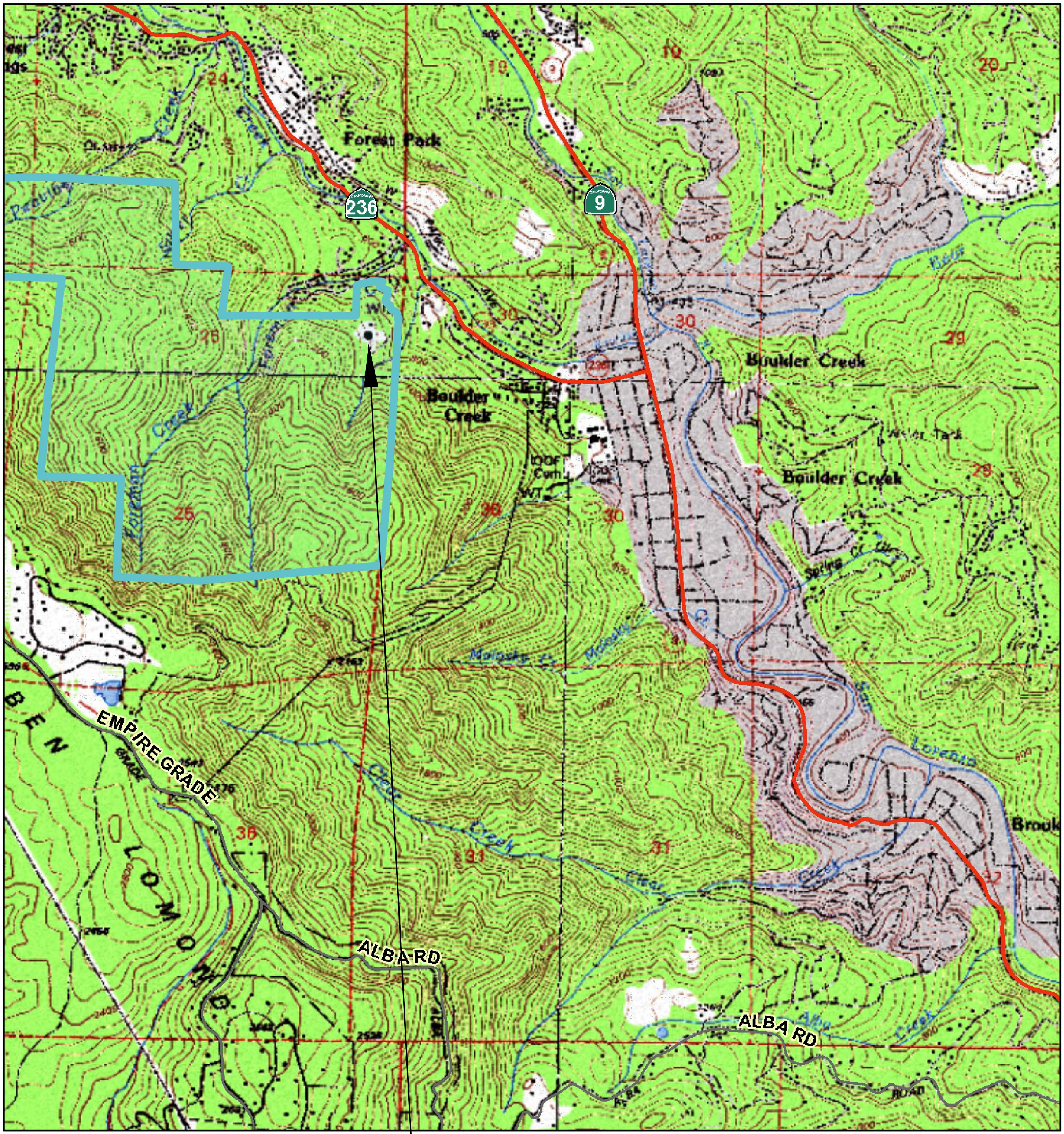
1. The recommendations of this report are based upon the assumption that the soil conditions do not deviate from those disclosed in the borings. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that planned at the time, our firm should be notified so that supplemental recommendations can be given.

2. This report is issued with the understanding that it is the responsibility of the owner, or his representative, to ensure that the information and recommendations contained herein are called to the attention of the Architects and Engineers for the project and incorporated into the plans, and that the necessary steps are taken to ensure that the Contractors and Subcontractors carry out such recommendations in the field. The conclusions and recommendations contained herein are professional opinions derived in accordance with current standards of professional practice. No other warranty express or implied is made.

3. The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they be due to natural processes or to the works of man, on this or adjacent properties. In addition, changes in applicable or appropriate standards occur whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated, wholly or partially, by changes outside our control. Therefore, this report should not be relied upon after a period of three years without being reviewed by a geotechnical engineer.

Project No. SC4090.1
6 August 2018

APPENDIX A



SITE LOCATION

1:24,297

0 0.2 0.4 0.8 mi

0 0.33 0.65 1.3 km

FROM:

SANTA CRUZ COUNTY GIS

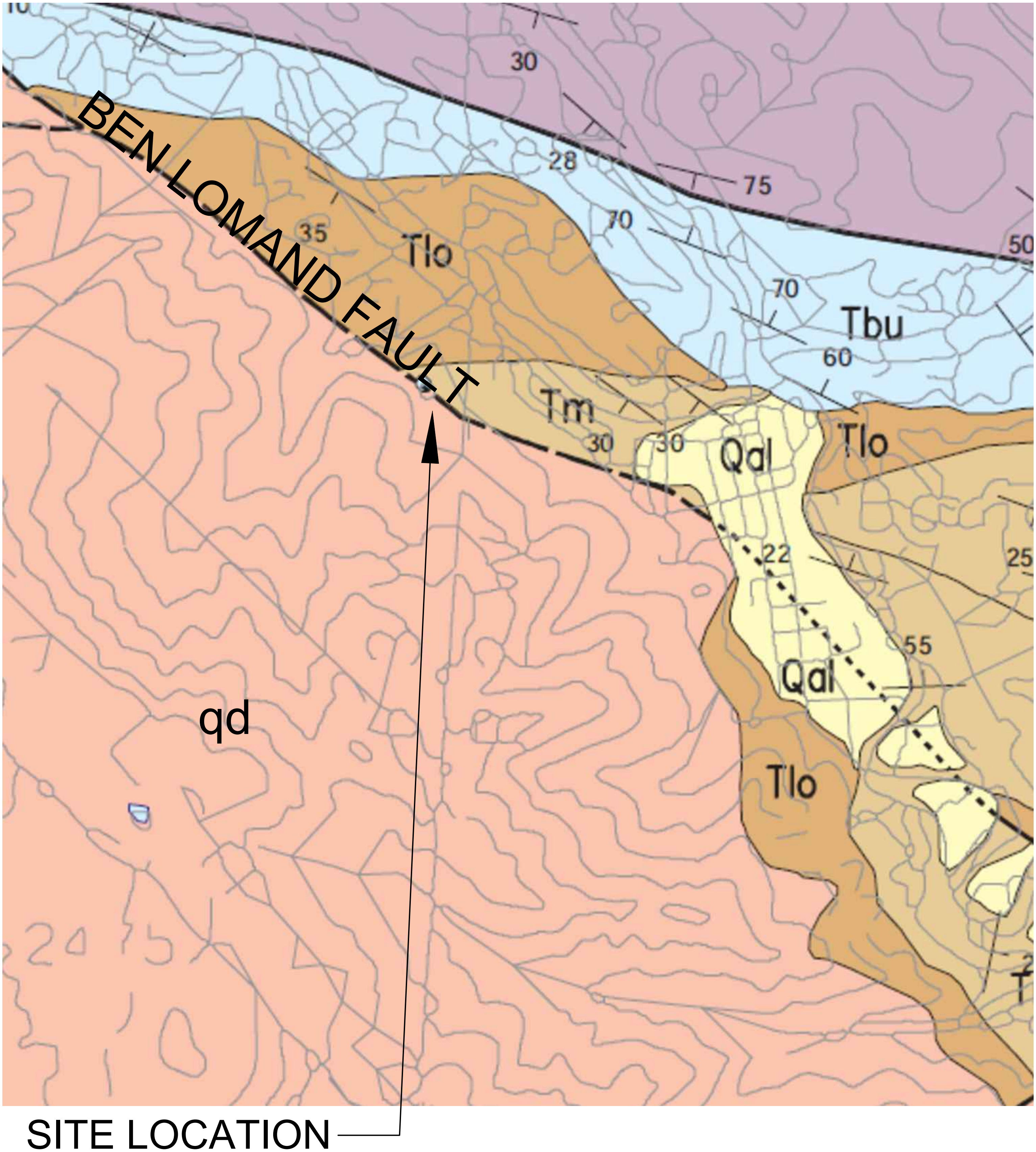
SITE VICINITY MAP
SLVWD - LYON TANK
BOULDER CREEK, CALIFORNIA
APN: 081-011-07

SCALE:	AS SHOWN
DRAWN BY:	AJB
DATE:	APR 2018
REVISED:	
JOB NO.	SC4090.1

HARO, KASUNICH & ASSOCIATES, INC.
GEOTECHNICAL AND COASTAL ENGINEERS
116 E. LAKE AVENUE, WATSONVILLE, CA 95076
(831) 722-4175

FIGURE NO. 1

SHEET NO.



KEY:

Tlo:	LOMPICO SANDSTONE (MIDDLE MIOCENE)
TM:	MONTEREY FORMATION (MIDDLE MIOCENE)
qd:	QUARTZ DIORITE (CRETACEOUS)

FROM:

GEOLOGIC MAP OF SANTA CRUZ COUNTY, CALIFORNIA
Compiled by
Earl E. Brabb
Digital Database Prepared by S. Graham, C. Wentworth, D. Knifong, R. Graymer and J. Blissenbach
1997

REGIONAL GEOLOGIC MAP
SLVWD - LYON TANK
BOULDER CREEK, CALIFORNIA
APN: 081-011-07

SCALE:	NTS
DRAWN BY:	AJB
DATE:	APR 2018
REVISED:	
JOB NO.	SC4090.1

HARO, KASUNICH & ASSOCIATES, INC.
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116 E. LAKE AVENUE, WATSONVILLE, CA 95076
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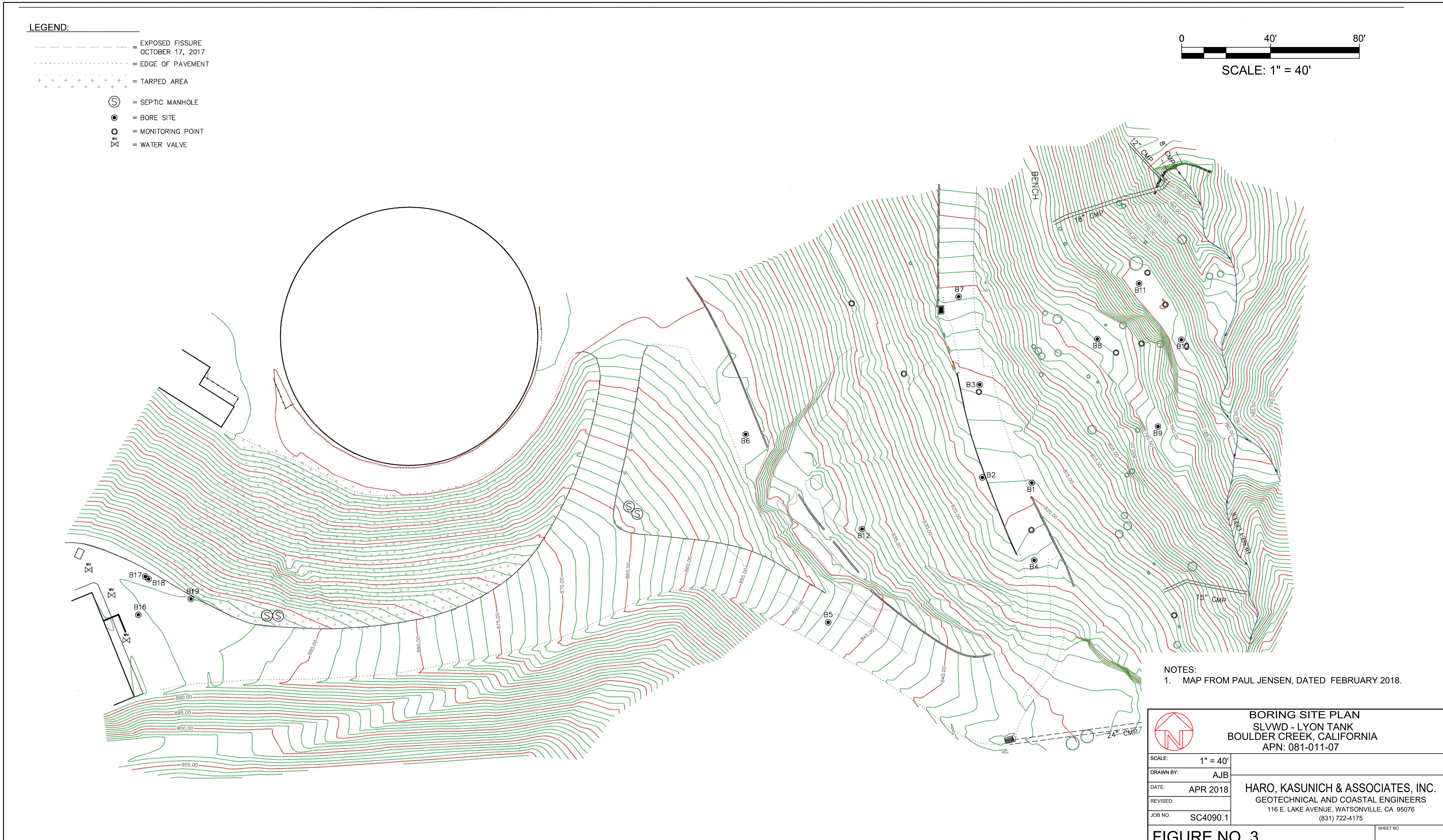
FIGURE NO. 2

SHEET NO.



LEGEND:
= EXPOSED FISSURE OCTOBER 17, 2017
= EDGE OF PAVEMENT
= TARPED AREA
= SEPTIC MANHOLE
= BORE SITE
= MONITORING POINT
= WATER VALVE
0
40'
80'
SCALE: 1" = 40'
NOTES:
1. MAP FROM PAUL JENSEN, DATED FEBRUARY 2018.
BORING SITE PLAN
SLVWD - LYON TANK
BOULDER CREEK, CALIFORNIA
APN: 081-011-07
SCALE: 1" = 40'
DRAWN BY: AJB
DATE: APR 2018
REVISED:
JOB NO. SC4090.1
HARO, KASUNICH & ASSOCIATES, INC.
GEOTECHNICAL AND COASTAL ENGINEERS
116 E. LAKE AVENUE, WATSONVILLE, CA 95076
(831) 722-4175
SHEET NO.
FIGURE NO. 3
B1
B2
B3
B4
B5
B6
B7
B8
B9
B10
B11
B12
B16
B17
B18
B19
BENCH
12" CMP
8" CMP
18" CMP
15" CMP
24" CMP
890.00
895.00
900.00
905.00

PRIMARY DIVISIONS			GROUP SYMBOL	SECONDARY DIVISIONS
COARSE GRAINED SOILS — MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS — MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (LESS THAN 5% FINES)	GW	Well graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.
		GRAVEL WITH FINES	GM	Silty gravels, gravel-sand-silt mixtures, non-plastic fines.
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines.
	SANDS — MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS (LESS THAN 5% FINES)	SW	Well graded sands, gravelly sands, little or no fines
			SP	Poorly graded sands or gravelly sands, little or no fines
		SANDS WITH FINES	SM	Silty sands, sand-silt mixtures, non-plastic fines.
			SC	Clayey sands, sand-clay mixtures, plastic fines.
FINE GRAINED SOILS — MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS — LIQUID LIMIT IS 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 IS GREATER THAN 50%		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils.

GRAIN SIZES

U.S. STANDARD SERIES SIEVE: 200, 40, 10, 4 — CLEAR SQUARE SIEVE OPENINGS: 3/4", 3", 12"

SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

RELATIVE DENSITY

SANDS AND GRAVELS	BLOWS PER FOOT*
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	OVER 50

CONSISTENCY

SILTS AND CLAYS	STRENGTH (TSF)**	BLOWS PER FOOT*
VERY SOFT	0 - ¼	0 - 2
SOFT	¼ - ½	2 - 4
FIRM	½ - 1	4 - 8
STIFF	1 - 2	8 - 16
VERY STIFF	2 - 4	16 - 32
HARD	OVER 4	OVER 32

SAMPLING METHOD

STANDARD PENETRATION TEST	T	
MODIFIED CALIFORNIA	L or M	
PITCHER BARREL	P	
SHELBY TUBE	S	
BULK	B	

H_2O

Final	▽
Initial	▼
Water level designation	

*Number of blows of 140 lb hammer falling 30 inches to drive a 2" O.D. (1⅜" I.D.) split spoon sampler (ASTM D-1586)
**Unconfined compressive strength in tons/ft^2 as determined by laboratory testing or approximated by the Standard Penetration Test (ASTM D-1586), pocket penetrometer, torvane, or visual observation.

KEY TO LOGS
SLVWD - LYON TANK
BOULDER CREEK, CALIFORNIA
APN: 081-011-07

SCALE:	NTS
DRAWN BY:	AJB
DATE:	APR 2018
REVISED:	
JOB NO.	SC4090.1

HARO, KASUNICH & ASSOCIATES, INC.
GEOTECHNICAL AND COASTAL ENGINEERS
116 E. LAKE AVENUE, WATSONVILLE, CA 95076
(831) 722-4175

FIGURE NO. 4

SHEET NO.

Haro, Kasunich & Associates, Inc.
Coastal and Geotechnical Engineers

Lyon Tank Slide

PROJECT NO. SC4090

LOGGED BY CG **DATE DRILLED** May 4, 2017 **BORING DIAMETER** 6" **BORING NO.** B-1

Depth, ft.	Sample No. and type	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0		Fill						
		Brown and gray medium Silty SAND with Clay binder, moist, medium dense	SM					
5		Fill, gray Granite SAND, moist, medium dense						
	1-1 (L)			26		105	12.7	
	1-2 (T)			13				
		Fill, gray Granitic Silty SAND, moist, medium dense						
10								
	1-3 (L)			14		104	15.1	
		Fill, Gray Silty Granitic SAND, wet, loose with plant roots	SM					
	1-4 (T)			6			16.6	
	1-5 (L)			11		94	22.6	
15	1-6 (T)	Fill, Gray coarse SAND, wet, loose		4				
	1-7 (T)	Fill, Gray Granite SAND with roots and wood from 16-17.5' wet, loose		4		101	15.7	
	1-8 (T)			7				
		Fill, gray SAND, coarse from 14.5 to 16 and 17.5 to 19'						
20		Landslide, brown Clayey SAND, saturated, loose	SC	1/18"				
	1-9 (T)						28.8	
	1-10(L)			6		87	26.6	
25	1-11(T)	Landslide, light brown Clayey SAND, moist, very loose		5			26.3	(1-11) Grain Size Analysis % Gravel = 1.5 % Sand = 62.9 % Fines = 35.6
30	1-12(L)	Landslide, light brown Clayey SAND with Gravels (much less Clay than 1-11, very moist, loose		9		103	23.5	
35								

HARO, KASUNICH AND ASSOCIATES, INC.

BY: dk **FIGURE NO.** 5

Haro, Kasunich & Associates, Inc.
Coastal and Geotechnical Engineers

Lyon Tank Slide

PROJECT NO. SC4090

LOGGED BY CG **DATE DRILLED** May 4, 2017 **BORING DIAMETER** 6" **BORING NO.** B-1

Depth, ft.	Sample No. and type	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
35	1-13(T)	Light brown & yellow brown Clayey SAND with Gravels, very moist, loose		8				
		Bottom Landslide (?)						
40	1-14(L)	Mottled orange brown CLAY, moist, very stiff	CL	20		97	28.8	
45		Harder drilling at 45'						
		Light brown SANDSTOE with orange stains, moist, very dense	BR					
50	1-15(T)	Light brown SANDSTONE with orange stains		50/3.5'				
		Boring terminated at 51.5 feet						
55								
60								
65								
70								

SuperLog CivilTech Software, USA www.civiltech.com File: C:\Superlog4\HKALOGS\SC4090 Lyon Tank Slide.log Date: 5/30/2018

HARO, KASUNICH AND ASSOCIATES, INC.

BY: dk **FIGURE NO.** 6

Lyon Tank Slide

Haro, Kasunich & Associates, Inc.
Coastal and Geotechnical Engineers

PROJECT NO. SC4090

LOGGED BY CG **DATE DRILLED** May 4, 2017 **BORING DIAMETER** 6" **BORING NO.** B-2

Depth, ft.	Sample No. and type	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0		Fill, mixed, Silty Clayey SAND with Gravels, moist, loose	SC					
5	2-1 (L)	Fill, mixed light brown Clayey SAND with Gravels, moist, loose		13		107	13.0	
	2-2 (T)	Increase in moisture from 6 1/2' to 8'		6			16.3	
		Loose and saturated from 8'						
10		Gray Clayey SAND with Gravels and roots, very moist to wet, loose	SC					
	2-3 (L)			2		92	23.9	
	2-4 (T)	Brown Clayey SAND with Gravels, very moist, loose	SC	3				
15	2-5 (L)	Light brown Clayey SAND with Gravels, very moist to wet, loose		7		91	29.6	
	2-6 (T)			5				
	2-7 (L)	Light brown Silty SAND with Gravels, very moist to wet, loose	SM	6		93	32.7	
20	2-8 (T)			7				
	2-9 (L)	Sandy CLAY, mosit, stiff, light brown Silty Granite SAND, wet, soft to medium stiff	CL	6		83	33.2	
	2-10(T)	Native, light brown CLAY very moist, firm-stiff (weathered bedrock?)	CL	8				
25	2-11(L)	Light brown CLAY, very moist, very stiff (weathered Bedrock?)		11		102	25.4	
	2-12(T)	Light brown Silty SANDSTONE, mosit, medium dense (weathered Bedrock)	SM	19				
30	2-13(T)	Very light brown Silty SANDSTONE with orange stains, moist, very dense	BR	50/4"			9.2	
		Boring terminated at 31.50 feet						
35								

SuperLog CivilTech Software, USA www.civiltech.com File: C:\Superlog4\HKALOGS\SC4090 Lyon Tank Slide.log Date: 5/30/2018

HARO, KASUNICH AND ASSOCIATES, INC.

BY: dk **FIGURE NO.** 7

Haro, Kasunich & Associates, Inc.
Coastal and Geotechnical Engineers

Lyon Tank Slide

PROJECT NO. SC4090

LOGGED BY CG **DATE DRILLED** May 4, 2017 **BORING DIAMETER** 6" **BORING NO.** B-2A

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0			Fill						
			Fill, brown Silty Clayey SAND with Gravels, moist, loose	SC					
5			Fill, mixed light brown Clayey SAND with Gravels						
			Increase in moisture from 6 1/2 to 8'						
	2A-1(L)		Gray Clayey SAND, very moist, very loose	SC	3				
10	2A-2(T)				2				
			Boring terminated at 11 feet						
15									
20									
25									
30									
35									

SuperLog CivilTech Software, USA www.civiltech.com File: C:\Superlog4\HKALOGS\SC4090 Lyon Tank Slide.log Date: 5/30/2018

HARO, KASUNICH AND ASSOCIATES, INC.

BY: dk **FIGURE NO.** 8

Haro, Kasunich & Associates, Inc.
Coastal and Geotechnical Engineers

Lyon Tank Slide

PROJECT NO. SC4090

LOGGED BY CG **DATE DRILLED** May 23, 2017 **BORING DIAMETER** 8" HS **BORING NO.** B-3

Depth, ft.	Sample No. and type	Soil Description	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	Misc. Lab Results
0	3-1 (B)	2" AC 10" AB Orange Gravelly SAND					23.2	
	3-2 (B)	Fill, Gray & orange Clayey SAND with Gravel, moist, medium dense	SC					
5		Water 5' at end of drilling						
	3-3 (L)	Fill, mixed orange brown Silty SAND with Clay & Gravels, very moist, loose	SM	19		116	11.0	
10		Water 10' first encountered (Weathered Granite)						
	3-4 (L)	Fill, Orange brown Clayey SAND, very moist, wet with Gravel, very loose	SC	9		104	17.1	
		Orange Gravelly SAND with Clay from 14' to 15'						
15		Wet, loose from 15' - 17.5'						
		Orange Clayey SAND, reddish brown decomposed wood from 19'-20'	SC					
20		Orange Clayey SAND, wet, loose	SC					
		Orange & brown SAND with Gravels from 23' - 25'	SM					
25		Orange & brown SAND and Gravelly SAND-Loose						
	3-5 (L)			7		92	25.6	
	3-6 (T)	Brown Sandy CLAY (weathered Granite) moist, firm-medium stiff	CL	7				
30		Orange brown Clayey SAND with seams of wet Gravelly (weathered Granite) SAND	SC					
		Very light brown SANDSTONE with orange stains & striations, moist, very dense	BR					
35	3-7 (T)			50/4"				(3-7) Grain Size Analysis %Gravel = 0.4 % Sand = 75.4 %Fine = 24.2

HARO, KASUNICH AND ASSOCIATES, INC.

BY: dk FIGURE NO. 9

Haro, Kasunich & Associates, Inc.
Coastal and Geotechnical Engineers

Lyon Tank Slide

PROJECT NO. SC4090

LOGGED BY CG **DATE DRILLED** May 23, 2017 **BORING DIAMETER** 8" HS **BORING NO.** B-4

Depth, ft.	Sample No. and type	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0		2" AC 6" AB orange Gravely SAND and gravy Clayey SAND						
	4-1 (B)							
	4-2 (B)	Orange Clayey SAND	SC					
		Water at 4' @ end of drilling						
5		Fill, gray & brown Clayey SAND						
	4-3 (L)			25		77	13.8	
	4-4 (T)			19				
10		Water first encountered		7				
	4-5 (L)	Gray Clayey SAND with Gravel and roots, water at 10'. wet. loose	SC			64	22.0	
	4-6 (T)			2				
15		Gray Clayey SAND and medium to coarse SAND with roots, wet		10				
	4-7 (L)					96	15.1	
	4-8 (T)	Clean grey SAND from 16.5 - 19' saturated, loose (Alluvial deposits)	SM	3			24.1	
	4-9 (T)			2				
		Native, gray Clayey SAND (weathered granite) wet. loose						
20	4-10(L)			3		81	32.4	
	4-11(T)							
25	4-12(T)	Gray & brown CLAY with thin seams of Gravel, wet, medium stiff	CL	7				
		Gray Silty & Clayey SAND, wet, loose						
30	4-13(T)	4" - 6" seams of orange coarse SAND & orange medium SAND, wet, medium dense (Alluvial deposits?)	SC	14			22.0	
35								

HARO, KASUNICH AND ASSOCIATES, INC.

BY: dk

Haro, Kasunich & Associates, Inc.
Coastal and Geotechnical Engineers

Lyon Tank Slide

PROJECT NO. SC4090

LOGGED BY CG **DATE DRILLED** May 23, 2017 **BORING DIAMETER** 8" HS **BORING NO.** B-4

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
35	4-14(T)		Interbedded 4"-6" thick seams of orange Clay, coarse SAND & medium Sand, wet, medium dense & very stiff (Alluvial deposits)	SC	19			22.0	
			Saturated gray Clayey SAND spoils from auger						
40	4-15(T)		Interbedded seams of medium to coarse light brown Sand, orange brown Clayey SAND, very moist to wet, medium dense to 45.5'		13				
45	4-16(T)		Orange decomposed Granite, mosit, very dense	BR/SM	57			9.2	(4-16) Grain Size Analysis % Gravel = 0.0 % Sand = 81.8 % Fines = 18.2
			Boring terminated at 46.5 feet						
50									
55									
60									
65									
70									

HARO, KASUNICH AND ASSOCIATES, INC.

BY: dk

FIGURE NO. 11

Haro, Kasunich & Associates, Inc.
Coastal and Geotechnical Engineers

Lyon Tank Slide

PROJECT NO. SC4090

LOGGED BY CG **DATE DRILLED** May 24, 2017 **BORING DIAMETER** 2" HS **BORING NO.** B-5

Depth, ft.	Sample No. and type	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0		Fill, brown weathered Granite, moist, medium dense	SC					
5		Native						
	5-1 (L)			7		102	11.8	
	5-2 (T)	Orange decomposed Granite, moist, loose	SN	5				
	5-3 (L)			10		99	13.8	(5-3) Direct Shear 0 = 36 C = 162 psf Ms = 20.3 Atterberg Limits LL = 26.48% PI = 4
10	5-4 (T)	Orange weathered Granite, moist, loose		8				
		Increase in drilling resistance from 11' - 15'						
15	5-5 (T)	Water at 15' after drilling Orange very weathered Granitic CLAY, moist, medium dense	CL	14				
20	5-6 (T)	Water on Supply Orange, very weathered Granitc, moist, medium dense	SC	10				
25		Orange, less weathered Granite, moise, loose		19				
	5-7 (L)					107	17.2	(5-7)Direct Shear 0 = 51 C = 232 psf Ms = 19.7%
	5-8 (T)	Orange weathered Granite, moist, medium dense		19				
30	5-9 (T)	Orange decomposed Granite, very moist, dense		42			94	
35								

HARO, KASUNICH AND ASSOCIATES, INC.

BY: dk **FIGURE NO.** 12

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Lyon Tank Slide

PROJECT NO. SC4090

LOGGED BY CG **DATE DRILLED** May 24, 2017 **BORING DIAMETER** 2" HS **BORING NO.** B-5

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
35	5-10(T)		Orange, brown decomposed Granite, medium dense to dense		27				
40	5-11(T)		Orange brown decomposed Granite, very moist, dense		39			14.2	
			Boring terminated at 41.50 feet						
45									
50									
55									
60									
65									
70									

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Lyon Tank Slide

PROJECT NO. SC4090

LOGGED BY CG **DATE DRILLED** May 24, 2017 **BORING DIAMETER** 8" HS **BORING NO.** B-6

Depth, ft.	Sample No. and type	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0		Fill Brown Silty SAND with Gravels, moist, medium dense	SM					
	6-1 (L)			22			10.7	
	6-2 (T)	Fill Orange brown Silty SAND, Granite, medium dense		13				
5	6-3 (L)			38		118	10.3	(6-3) Direct Shear 0 = 47 C = 463 psf Ms = 15.1%
	6-4 (T)			19				
10	6-5 (T)	Fill Orange brown decomposed Silty SAND, Granite, moist		20		99	10.9	
15	6-6 (T)	Fill Orange brown decomposed Granite, moist, medium dense		14			14.1	
20	6-7 (T)	Fill Orange brown decomposed Grante, moist, medium dense		11			14.0	
		Native (?) Gray weathered Granite, very moist, loose	SM					
25	6-8 (T)	Gray, very weathered decomposed Granite, very moist, loose		7				
	6-9 (L)			11		89	26.8	(6-9) Direct Shear 0 = 37 C = 611 psf
30	6-10(T)	Orange, very weathered Granite, very moist, loose		4				
	6-11(T)			50 2 1/2				
		Light brown SANDSTONE (Lompico Sandstone) with orange stains, mover, very dense	BR					
		Boring terminated at 33.0 feet						
35								

HARO, KASUNICH AND ASSOCIATES, INC.

BY: dk **FIGURE NO.** 14

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Lyon Tank Slide

PROJECT NO. SC4090

LOGGED BY CG **DATE DRILLED** May 24, 2017 **BORING DIAMETER** 8" HS **BORING NO.** B-7

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0			2" AC						
			Fill, Mixed orange brown, olive brown & gray weathered Granite, moist, loose to medium dense	SM					
	7-1 (L)				17				
5			Olive brown weathered Granite, moist						
	7-2 (L)		Fill Mixed orange brown & gray weathered Granite		26		122	11.3	
10	7-3 (T)				12				
			Fill, Orange brown weathered Granite, very moist, medium dense						
15	7-4 (T)		Fill, Orange brown weathered Granite, very moist, medium dense		12			12.7	
			Easier drilling from 17' - 20'						
20	7-5 (T)		Fill Orange brown very weathered Granite, very moist, loose	SM	3				
25	7-6 (L)		Filter Fabric Orange gravelly SAND, very moist, loose		12			4.1	
			Very hard drilling at 27'	BR					
30	7-7 (L)		Light brown SANDSTONE with orange stains, moist, very dense		50/2"		114	12.4	
			Boring terminated at 30 feet						
35									

HARO, KASUNICH AND ASSOCIATES, INC.

BY: dk

Lyon Tank Slide

PROJECT NO. SC4090

LOGGED BY ________ **DATE DRILLED** ________ **BORING DIAMETER** ________ **BORING NO.** B-8

Depth, ft.	Sample No. and type	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0		Native						
	8-1 (L)	Yellow brown fine to medium SAND loose to medium dense from 0-3 1/2'	SM	11		118	3.3	
	8-2 (T)			12				
		SAND seam at 4' (decomposed Granite						
5	8-3 (T)	Dark yellow brown Silty SAND with Clay, mica & occassional Gravels, moist loose		4				
	8-4 (L)	Dark yellow		4				
10	8-5 (L)	Brown & gray Silty SAND with mica & angular coarse SAND, very moist, loose (decomposed Granite)	SM	6		106	20.0	(8-5) Direct Shear 0 = 42 C = 358 psf Ms = 21.0%
		Hole caved to 12'						
15	8-6 (L)	Brown with gray pockets Silty SAND with mica and small roots, very moist, very loose	SM	8				(8-6) Direct Shear 0 = 40 C = 0
20	8-7 (T)	Gray SANd with Silt & Gravels, very moist, loose		9				
		Gray Clayey SAND in Auger cuttings from 24-25'	SC					
25	8-8 (L)	Gray Clayey SAND with Gravels, very moist - wet, loose		13				
	8-9 (T)			6				
		Gray Clayey medium to coarse SAND with occassional 1/2" to 1" diameter angular Gravels, wet, loose						
30	8-10 (L)	Buried piece of decomposed wood at 30', grading more Clayey from 30' - 35'	SC	17				
35								

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Lyon Tank Slide

PROJECT NO. SC4090

LOGGED BY ______ **DATE DRILLED** ______ **BORING DIAMETER** ______ **BORING NO.** B-8

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
35	8-11(L)		Gray brown Sandy CLAY, wet, very soft	SC	1				No recovery
			Harder drilling at 38'						
40	8-12(T)		Light brown SANDSTONE with orange Gravels, moist, very dense	BR	50/4"				
45	8-13(T)		Light brown SANDSTONE with orange Gravels, moist, very dense		50/2"				
			Boring terminated at 46.5 feet						
50									
55									
60									
65									
70									

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HARO, KASUNICH AND ASSOCIATES, INC.

BY: dk **FIGURE NO.** 17

Lyon Tank Slide

Haro, Kasunich & Associates, Inc.

PROJECT NO. SC4090

LOGGED BY CG **DATE DRILLED** July 24, 2017 **BORING DIAMETER** 8" HS **BORING NO.** B-9

Depth, ft.	Sample No. and type	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0		Orange brown Silty medium to coarse SAND with mica, moist, loose (decomposed granite)	SM					
	9-1 (L)			15			2.9	
	9-2 (T)			6				
5	9-3 (L)	Orange brown Silty SAND with Gravels & Mica, moist, loose (decomposed granite)	SM	11			15.0	
	9-4 (T)			5				
10	9-5 (T)	Water at end of drilling Buried Decomposed Wood from 10' - 11.5'		3				
	9-6 (L)	Orange brown Clayey Silty SAND/Sandy CLAY with wood, mica and Gravels (weathered decomposed granite)	SC	8				
15	9-7 (T)	2" soil & wood debris in sample Buried decomposed wood from 15' - 16.5'		6			41.6	
20	9-8 (T)	Orange and brown Clayey SAND weathered Granite with Mica, wet, medium dense	SC	11		118		
25	9-9 (T)	Orange brown Sandy CLAY, wet, loose (very weathered Granite shale)	CL	6			26.4	
30	9-10(L)	Weathered Granite (intact) very moist, loose to medium dense		22		111	18.8	
		Harder drilling at 32'						
35								

HARO, KASUNICH AND ASSOCIATES, INC.

BY: dk **FIGURE NO. 18**

Lyon Tank Slide

PROJECT NO. SC4090

LOGGED BY CG **DATE DRILLED** July 24, 2017 **BORING DIAMETER** 8" HS **BORING NO.** B-9

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
35	9-11(T)		Orange & black medium to coarse SAND medium to coarse with Silty CLAY less weahtered decomposed Granite, very moist, dense	SC	40				
			Brown Silty SAND	SM					
40	9-12(T)		Decomposed Granite, wet, very dense		58			15.4	
			Boring terminated at 41.5 feet						
45									
50									
55									
60									
65									
70									

HARO, KASUNICH AND ASSOCIATES, INC.

BY: dk **FIGURE NO.** 19

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Lyon Tank Slide

PROJECT NO. SC4090

LOGGED BY CG **DATE DRILLED** July 25, 2017 **BORING DIAMETER** 8" HS **BORING NO.** B-10

Depth, ft.	Sample No. and type	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0		Dark brown Clayey SAND with Gravel & roots, very moist, loose	SC					
	10-1(T)			4			18.6	
	10-2(L)			6				
5		Dark brown Clayey SAND with Gravels						
		Water at 1:30 pm						
		Water and end of drilling 10:32 am						
10	10-3(L)	Orange Clayey Gravelly SAND, very moist, loose (decomposed granite)		17		104	12.4	
15	10-4(T)	Gray Gravelly SAND (decomposed grainte) wet, medium dense	SC	16			14.6	
20	10-5(T)			4			19.6	
		Gray Clayey fine SAND with angular Gravels (slide debris), loose to medium dense						
	10-6 (T)			11				
25	10-7(T)	Gray Clayey SAND with Gravels & wood fragment (slide debris?) wet, medium dense		16			21.6	
30	10-8(T)	Orange decomposed Granite, very moist, medium dense, grading to dense decomposed Granite from 30' - 35'		12				
	10-9(T)	Orange decomposed Granite with black specs, very moist, dense		47			18.1	
35								

HARO, KASUNICH AND ASSOCIATES, INC.

BY: dk **FIGURE NO.** 20

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Lyon Tank Slide

PROJECT NO. SC4090

LOGGED BY CG **DATE DRILLED** July 25, 2017 **BORING DIAMETER** 8" HS **BORING NO.** B-11

Depth, ft.	Sample No. and type	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0								
	11-1(L)	Light orange brown Silty SAND with large root, moist, medium dense (slide material)	SM	26		81	10.8	(11-1) Grain Size Analysis % Gravel = 5.2 % Sand = 67.0 % Fines = 27.8
5	11-2(T)	Dark brown Clayey medium to coarse SAND with small and large roopts, moist - very moist, loose	SC	4				
	11-3(L)			14				
		Gray Silty medium to coarse SAND with large wood fragment, medium dense	SM					
10	11-4(L)			23		115	10.7	(11-4) Grain Size Analysis % Gravel = 13.5 % Sand = 73.2 % Fines = 13.3
		Harder drilling (steady drilling) orange brown SAND with black fleck Decomposed Granite, very moist, dense						
15	11-5(T)			41				
		Gray medium to coarse SAND Decomposed Granite, moist, medium dense to dense						
		Gray Sandy CLAY, very moist, stiff	CL					
20	11-6(T)			12			26.1	
25	11-7(L)	Orange brown Sandy CLAY, very moist, firm (old slide material), medium stiff	CL	11		104	20.3	(11-7) Direct Shear 0 = 32 C = 367 psf Ms = 22.0%
30		Harder driling at 30'		29				
	11-8(T)	Orange SAND with Silty and black flecks (Decomposed Granite) very moist, medium dense, bands of orange gray brown and gray coarser Sand from 7' to 7 1/2' ??	SM				19.0	
	11-9(T)			50/3"				
35		Orange decomposed granite, damp, very dense	BR					

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HARO, KASUNICH AND ASSOCIATES, INC.

BY: dk

FIGURE NO. 21

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Lyon Tank Slide

PROJECT NO. SC4090

LOGGED BY CG **DATE DRILLED** July 25, 2017 **BORING DIAMETER** 8" H **BORING NO.** B-12

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0			Fill (Landslide Material)						
	12-1(L)		Orange brown Silty SAND with Gravels, moist, loose	SM	15		89	6.2	
5	12-2(T)		Fill (Landslide Materials) Orange brown Silty SAND with Gravels, moist, very loose	SM					Sample deflecte by rock at 5'
10	12-3(T)		Orange brown Silty SAND with Gravels, moist, loose	SM	5		101	17.1	Refusal at 12-13' Gray Granite Rock & Galvanized Wire (Gabion Basket)
	12-4(L)		Gray brown Clayey SAND	SC	50/5"				
15	12-5(L)		Fill Dark orange brown Clayey SAND with mica				81		
20	12-6(L)		Orange Sandy CLAY, stiff	CL	27				
	12-7(T)		Orange very weathered Granite, very moist, loose		8			18.3	
25	12-8(L)		Orange Clay very weathered Granite, very moist, soft		7		97	22.8	(12-8) Direct Shear 0 = 45 C = 292 psf Ms = 24.4%
	12-9(T)		Orange Sandy CLAY (very weathered Granite)		7				
30			Orange Sandy CLAY (very weathered Granite) wet, soft						
	2-10(L)		Orange less weathered Granite, wet, hard		88				
	2-11(T)		Light brown SANDSTONE with orange bands, moist, very dense		50/6"			13.1	
			Boring terminated at 32.5 feet						
35									

HARO, KASUNICH AND ASSOCIATES, INC.

BY: dk **FIGURE NO.** 22

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Lyon Tank Slide

PROJECT NO. SC4090

LOGGED BY CG **DATE DRILLED** November 22, 2017 **BORING DIAMETER** 8" **BORING NO.** B-13

Depth, ft.	Sample No. and type	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0		2" AC 5" AB						
		Fill Orange brown Silty SAND, moist, loose, very loose from 2' - 5'						
5	13-1 (L)			2		108	13.0	
	13-2 (T)	Fill, gray Silty SAND with Clay/Clayey SAND & Gravels, moist, medum dense	SM/SC	19				
10	13-3 (L)	Gray Silty SAND with Clay & Gravels, moist, medium dense	SM	44		116	15.6	
15	13-4 (L)	Gray Silty SAND with Clay, moist, medium dense	SM	33		115	13.3	
20	13-5 (L)	Native, gray Silty, Clayey SAND/Silty fine Sand with Clay (weathered Granite)	SC	40		125	11.2	(13-5) Grain Size Analysis % Gravel = 2.6 % Sand = 61.4 % Fines = 35.8
		Harder drilling @ 23 feet Gray granitic SAND, wet, very dense						
25	13-6 (L)	Water at 26' at end of drilling Slow drilling from 25' to 27'	SC	56/2"				
30	13-7 (T)	Gray granitic SAND with angular Gravel, wet, dense		41			15.1	
	13-8 (T)			50/4"			13.0	
		Boring terminated at 32.5 feet						
35								

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HARO, KASUNICH AND ASSOCIATES, INC.

BY: dk **FIGURE NO.** 23

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Lyon Tank Slide

PROJECT NO. SC4090

LOGGED BY CG **DATE DRILLED** October 22, 2017 **BORING DIAMETER** 6" **BORING NO.** B-14

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0			Fill						
	14-1 (L)		Orange Silty SAND with Clay, moist, very loose from 2' - 5 1/2'	SM	10				
	14-2 (T)				2				
5	14-3 (L)				2		101	14.8	(14-3) Atterberg Limits LL = 22,7% PI = 7%
	14-4 (T)		Fill Gray brown Silty SAND with Clay & Gravel, moist, medium dense	SC	17		98.5		
			Boring terminated at 7.0 feet						
10									
15									
20									
25									
30									
35									

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HARO, KASUNICH AND ASSOCIATES, INC.

BY: dk **FIGURE NO.** 24

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Lyon Tank Slide

PROJECT NO. SC4090

LOGGED BY CG **DATE DRILLED** November 22, 2017 **BORING DIAMETER** 6" **BORING NO.** B-15

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0			Fill Orange brown Silty SAND with Clay & Gravels, moist, loose	SM/SC					
	15-1 (L)				15				
	15-2 (T)				5				
5	15-3 (L)		Fill Light brown (white) SAND, moist, medium dense	SP	45		110	12.9	(15-2) Grain Size Analysis % Gravel = 3.3 % Sand = 66.0 % Fines = 30.7
	15-4 (T)				29			17.4	
			Fill Mixed gray & orange Silty SAND with Clay & Gravels, moist	SM					
10	15-5 (T)				24			12.6	
15	15-6 (T)		Fill Mixed orange & gray brown Silty SAND, moist, medium dense - dense		30			11.6	
			Native Gray Silty Clayey SAND, moist, medium dense	SC					
20	15-7 (T)				26			14.4	
25	15-8 (T)		Gray Silty SAND with Clay, moist, medium dense	SM	22			13.8	
30	15-9 (T)		Gray Silty SAND, moist, dense	SM	48				
			Boring terminated at 31.5 feet						
35									

HARO, KASUNICH AND ASSOCIATES, INC.

BY: dk **FIGURE NO.** 25

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Lyon Tank Slide

PROJECT NO. SC4090

LOGGED BY CG **DATE DRILLED** November 22, 2017 **BORING DIAMETER** 6" **BORING NO.** B-16

Depth, ft.	Sample No. and type	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0		Fill Mixed gray Silty SAND with Gravel, moist, medium	SM					
	16-1 (L)			28		114	13.0	(16-1) Grain Size Analysis % Gravel = 3.0 % Sand = 61.9 % Fines = 35.1
	16-2 (T)			22			13.2	
5		Fill(?) Orange gray Clayey SAND with Gravels, moist, medium dense	SC					
	16-3 (L)			41				(16-3) Grain Size Analysis % Gravel= 0.9 % Sand = 58.8 % Fines = 40.3
	16-4 (T)			27			12.8	
10		Mixed orange & gray Clayey SAND with Gravel, moist, medium dense						
	16-5 (T)			25			13.3	(16-4) Atterberg Limits LL = 24.1% PI = 9%
15	16-6 (T)	Mixed orange & gray Silty SAND, moist, medium dense		23			12.9	
20	16-7 (T)	Gray Silty CLAY with Sand, moist, medium dense	ML-CL	24				(16-7) Atterberg Limits ll = 24.2% PI
		Boring terminated at 21.5 feet						
25								
30								
35								

HARO, KASUNICH AND ASSOCIATES, INC.

BY: dk **FIGURE NO.** 26

Gravel Content:	**5.2%**
Sand Content:	**67.0%**
Fines Content	**27.8%**
Cumulative Sum	**100.0%**

Sample Description: Brown Clayey SAND
Group Symbol: SC

Haro, Kasunich & Associates, Inc.
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116 East Lake Avenue, Watsonville, California
(831) 722-4175 ~ Fax (831) 722-3202

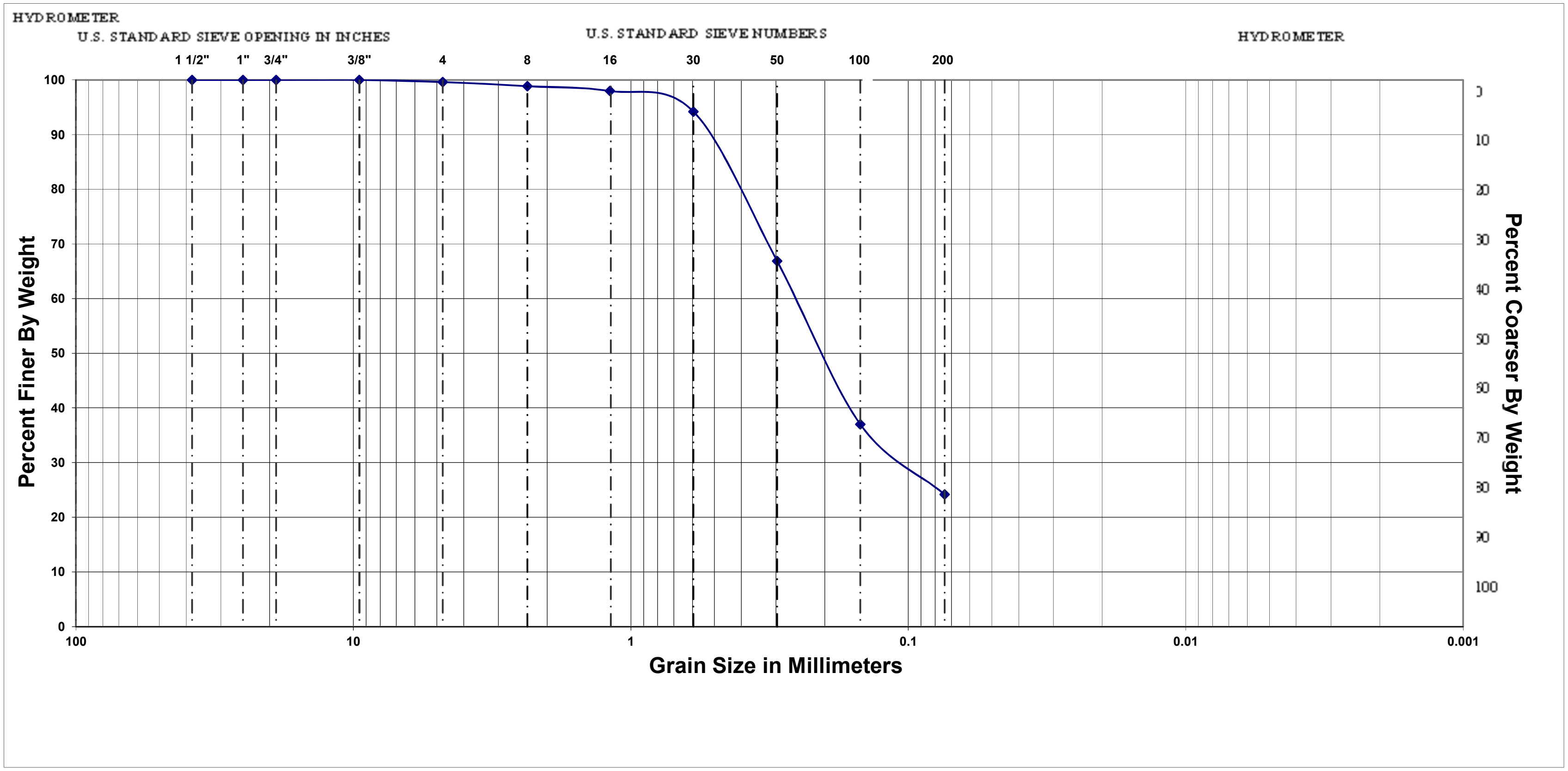
D60	
D30	
D10	

HKA Project No: SC 4090.1
Sample No: 11-1-1
Date: January 23, 2018

GRAIN SIZE ANALYSIS

SLVWD LYON TANK

Figure No.



Gravel Content:	**0.4%**
Sand Content:	**75.4%**
Fines Content	**24.2%**
Cumulative Sum	**100.0%**

Sample Description: Light Brown Silty SAND
Group Symbol: SM

Haro, Kasunich & Associates, Inc.
Coastal and Geotechnical Engineers
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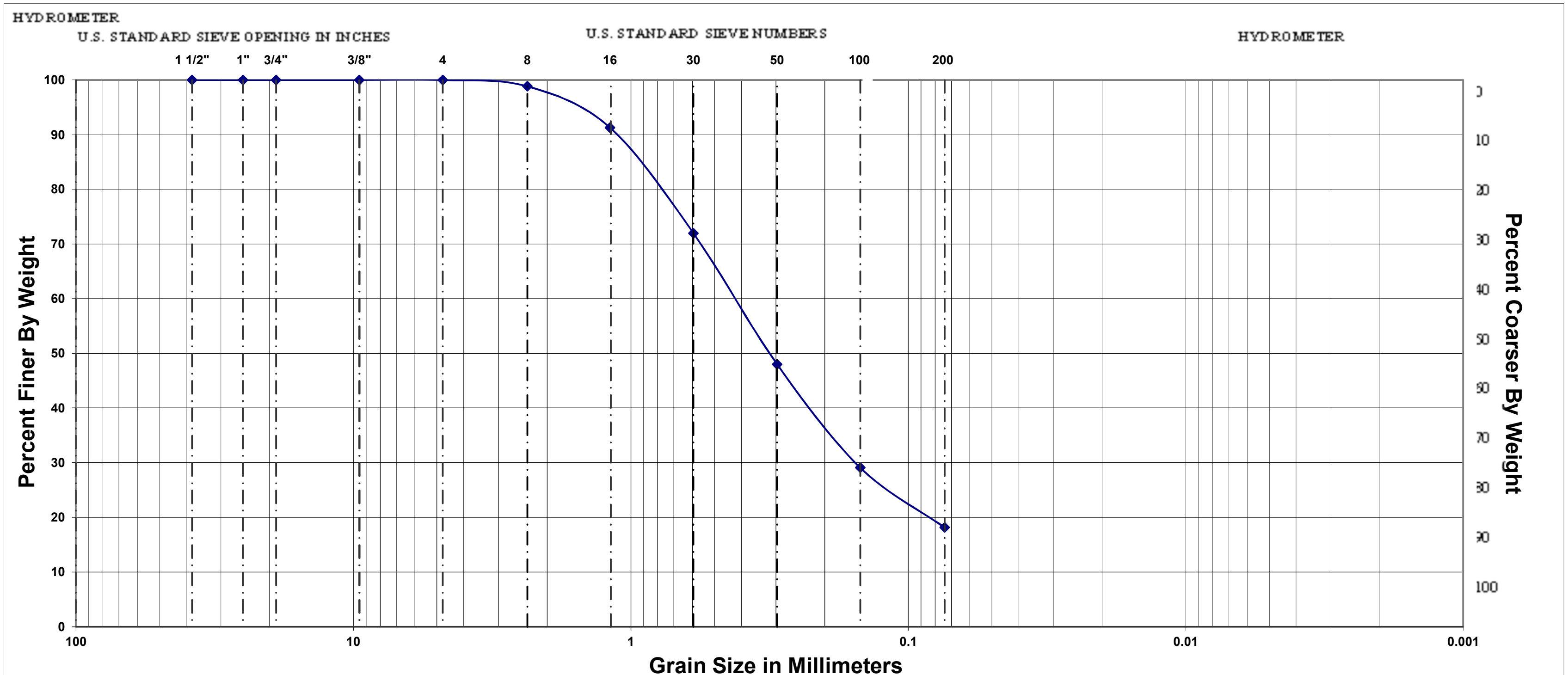
D60		HKA Project No:	SC 4090.1
D30		Sample No:	3-7
D10		Date:	1/25/17

GRAIN SIZE ANALYSIS

SLVWD LYON TANK



Gravel Content:	**0.0%**
Sand Content:	**81.8%**
Fines Content	**18.2%**
Cumulative Sum	**100.0%**

Sample Description: Golden Brown Silty SAND
Group Symbol: SM

D60		HKA Project No:	SC 4090.1
D30		Sample No:	4-16
D10		Date:	1/25/18

GRAIN SIZE ANALYSIS

SLVWD LYON TANK

Haro, Kasunich & Associates, Inc.
Coastal and Geotechnical Engineers
116 East Lake Avenue, Watsonville, California
(831) 722-4175 ~ Fax (831) 722-3202

Gravel Content:	**5.2%**
Sand Content:	**67.0%**
Fines Content	**27.8%**
Cumulative Sum	**100.0%**

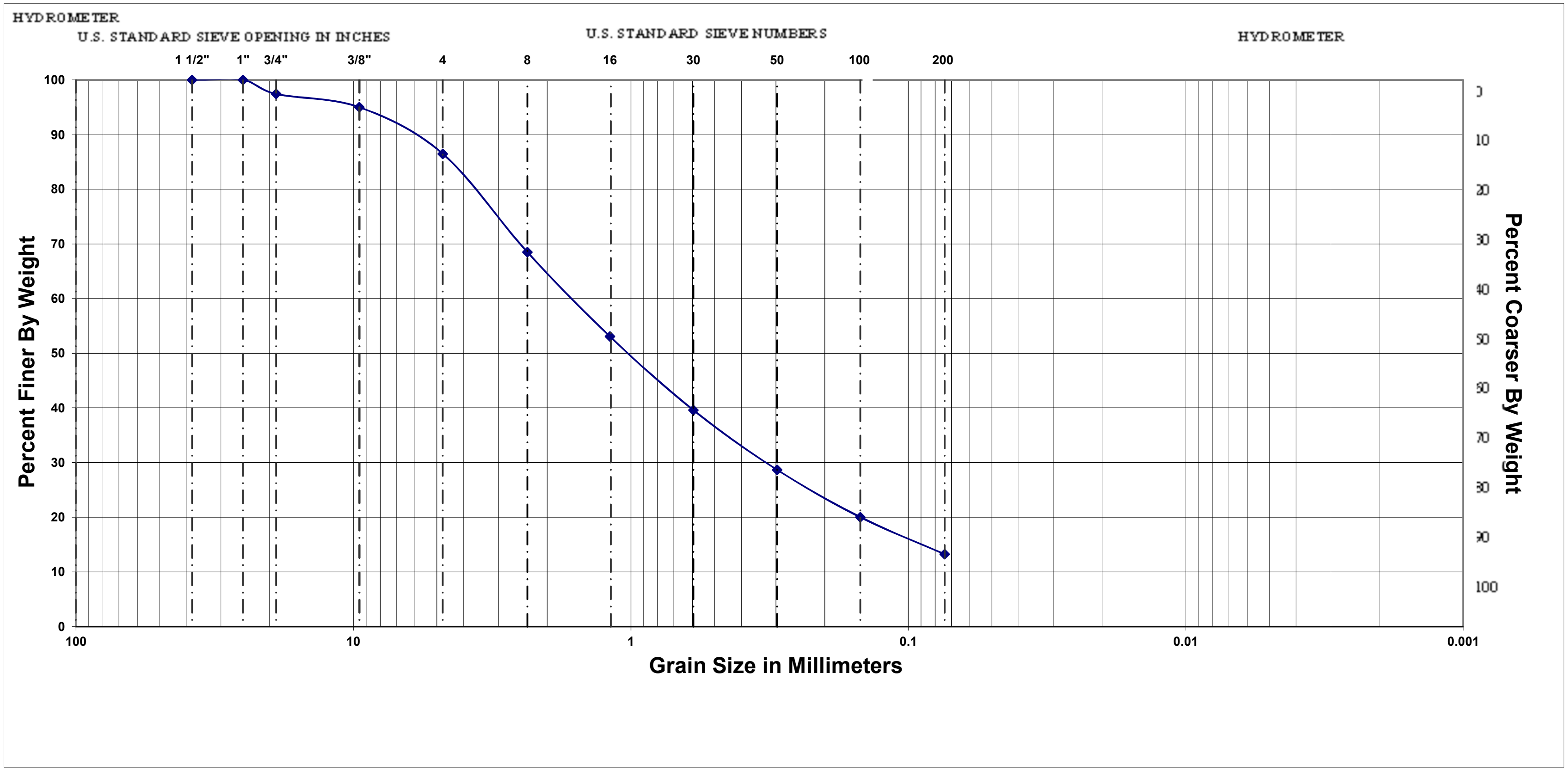
Sample Description: Brown Clayey SAND
Group Symbol: SC

Haro, Kasunich & Associates, Inc.
Coastal and Geotechnical Engineers
116 East Lake Avenue, Watsonville, California
(831) 722-4175 ~ Fax (831) 722-3202

D60		HKA Project No:	SC 4090.1
D30		Sample No:	11-1-1
D10		Date:	January 23, 2018

GRAIN SIZE ANALYSIS

SLVWD LYON TANK



Gravel Content:	**13.5%**
Sand Content:	**73.2%**
Fines Content	**13.2%**
Cumulative Sum	**100.0%**

Sample Description: **Brown Silty Sand w/ Gravel**
Group Symbol: **SM**

Haro, Kasunich & Associates, Inc.
Coastal and Geotechnical Engineers

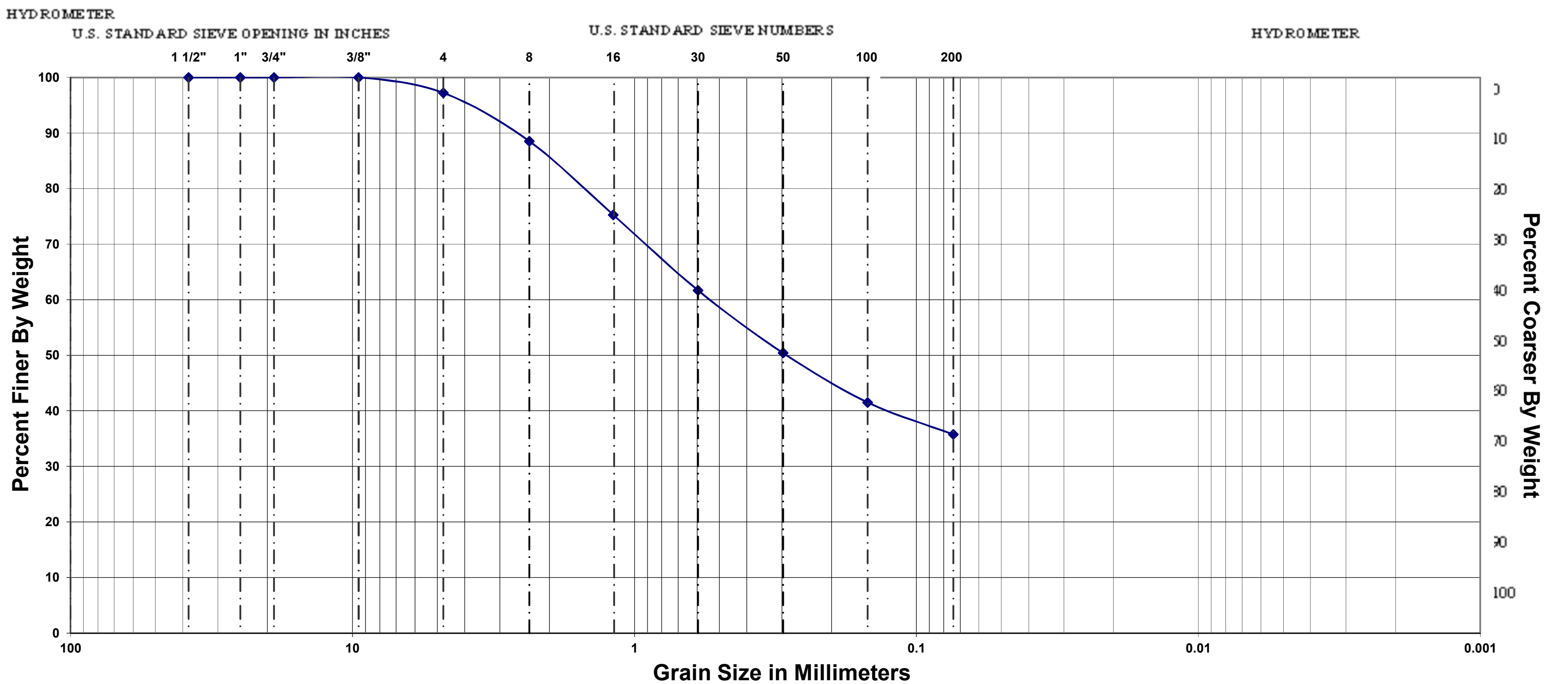
116 East Lake Avenue, Watsonville, California
(831) 722-4175 ~ Fax (831) 722-3202

D60	
D30	
D10	

HKA Project No:	SC 4090.1
Sample No:	11-4-2
Date:	1/22/18

GRAIN SIZE ANALYSIS

SLVWD LYON TANK



Gravel Content:	**2.8%**
Sand Content:	**61.4%**
Fines Content	**35.8%**
Cumulative Sum	**100.0%**

Sample Description: Dark Grey Clayey Sand
Group Symbol: SC

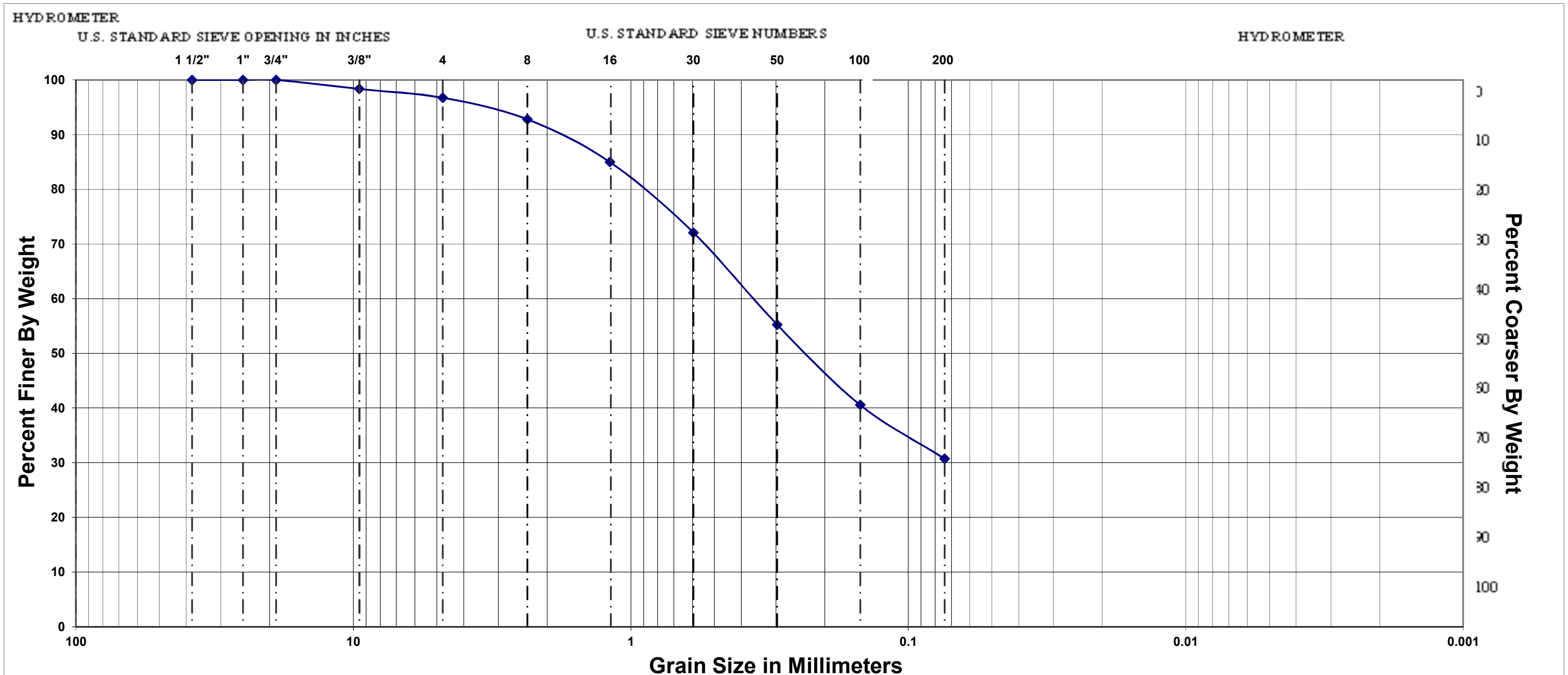
Haro, Kasunich & Associates, Inc.
Coastal and Geotechnical Engineers

116 East Lake Avenue, Watsonville, California
(831) 722-4175 ~ Fax (831) 722-3202

D60		HKA Project No:	SC 4090.1
D30		Sample No:	13-5-1
D10		Date:	1/24/18

GRAIN SIZE ANALYSIS

SLVWD LYON TANK



Gravel Content:	**3.3%**
Sand Content:	**66.0%**
Fines Content	**30.7%**
Cumulative Sum	**100.0%**

Sample Description: Brown Clayey Sand
Group Symbol: SC

Haro, Kasunich & Associates, Inc.
Coastal and Geotechnical Engineers

116 East Lake Avenue, Watsonville, California
(831) 722-4175 ~ Fax (831) 722-3202

D60		HKA Project No:	SC 4090.1
D30		Sample No:	15-2
D10		Date:	1/24/18

GRAIN SIZE ANALYSIS

SLVWD LYON TANK

Gravel Content: 3.0%
Sand Content: 61.9%
Fines Content 35.1%
Cumulative Sum 100.0%

Sample Description: Olive Brown Clayey Sand
Group Symbol: SC

Haro, Kasunich & Associates, Inc.
Coastal and Geotechnical Engineers

116 East Lake Avenue, Watsonville, California
(831) 722-4175 ~ Fax (831) 722-3202

D60		HKA Project No:	SC 4090.1
D30		Sample No:	16-1-1
D10		Date:	1/22/18

GRAIN SIZE ANALYSIS

SLVWD LYON TANK

Gravel Content:	**0.9%**
Sand Content:	**58.8%**
Fines Content	**40.3%**
Cumulative Sum	**100.0%**

Sample Description: Olive Brown Clayey Sand
Group Symbol: SC

D60	
D30	
D10	

HKA Project No: SC 4090
Sample No: 16-3-1
Date: 11/27/17 - 12/18/17

GRAIN SIZE ANALYSIS

SLVWD LYON TANK

Haro, Kasunich & Associates, Inc.
Coastal and Geotechnical Engineers
116 East Lake Avenue, Watsonville, California
(831) 722-4175 ~ Fax (831) 722-3202

Liquid Limit:	26.5
Plastic Limit:	22.7
Plasticity Index:	3.8

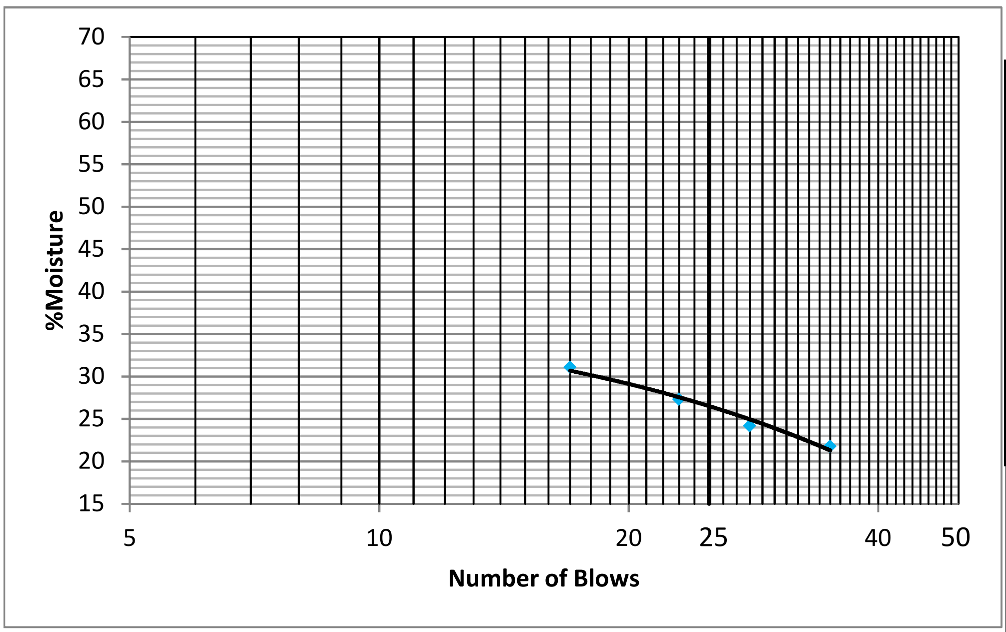
PI
4

HARO, KASUNICH AND ASSOCIATES, INC.
CONSULTING GEOTECHNICAL & COASTAL ENGINEERS

File N◦	SC 4090.1
Sample N◦	5-3-1
Date:	1/25/2017
By:	RC

P.I. SOIL TEST

	PLASTIC LIMIT				LIQUID LIMIT NUMBER OF BLOWS			
Determination	1	2	3	4	35	28	23	17
Tare N◦	14	3	10		6e	4f	5e	1c
Gross Wet WT.	13.57	15.32	16.70		12.24	10.42	13.53	11.49
GrossDry WT.	13.10	14.54	15.56		10.80	9.20	11.52	9.76
Tare WT.	11.07	11.19	11.02		4.20	4.16	4.17	4.20
NET DRY WT.	2.03	3.35	4.54	0.00	6.60	5.04	7.35	5.56
WT. OF Water	0.47	0.78	1.14	0.00	1.44	1.22	2.01	1.73
% Moisture	23.15	23.28	25.11	#DIV/0!	21.82	24.21	27.35	31.12



Sample #	5-3-1
Ht. of Sample	bag
Tare	4
Gross Wet Wt	282.5
Gross Dry Wt.	261.8
Tare Wt.	109.8
Net Dry Wt.	152.0
Wt. Of Water	20.7
% Moisture	13.6%
Dry Density	#VALUE!
Gold and Light Brown	
Elastic silt	
Group Symbol	SM

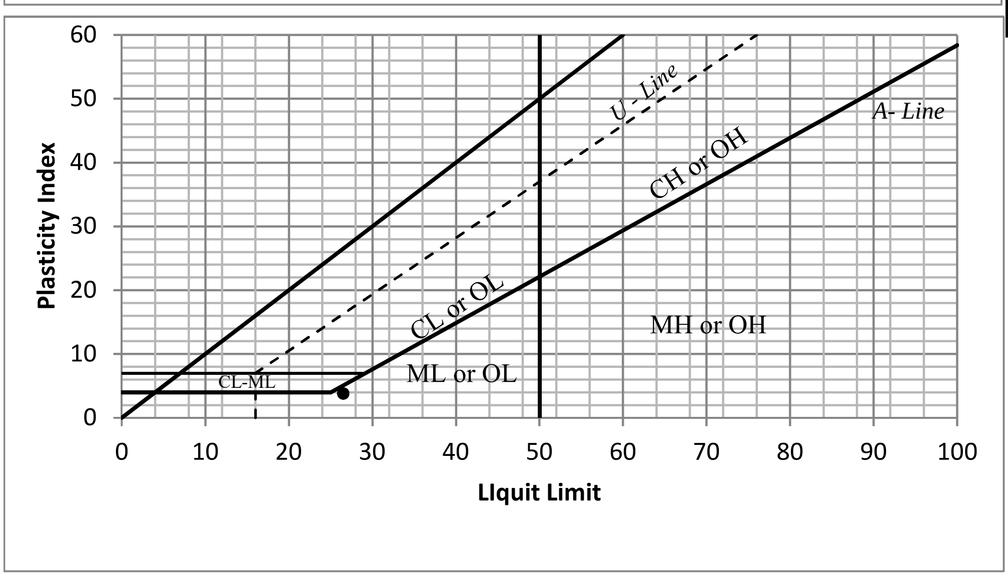


FIGURE NO. 36

Liquid Limit:	22.7
Plastic Limit:	15.8
Plasticity Index:	7.0

PI
7

HARO, KASUNICH AND ASSOCIATES, INC.
CONSULTING GEOTECHNICAL & COASTAL ENGINEERS

File N◦	SC 4090.1
Sample N◦	14-3-1
Date:	1/25/2018
By:	RC

P.I. SOIL TEST

	PLASTIC LIMIT				LIQUID LIMIT NUMBER OF BLOWS			
Determination	1	2	3	4	34	28	23	16
Tare N◦	22	31	12		1a	3c	5e	4e
Gross Wet WT.	14.91	13.62	15.87		12.05	14.41	11.10	11.12
GrossDry WT.	14.41	13.26	15.26		10.77	12.58	9.82	9.66
Tare WT.	11.20	11.00	10.97		4.21	4.16	4.16	4.19
NET DRY WT.	3.21	2.26	4.29	0.00	6.56	8.42	5.66	5.47
WT. OF Water	0.50	0.36	0.61	0.00	1.28	1.83	1.28	1.46
% Moisture	15.58	15.93	14.22	#DIV/0!	19.51	21.73	22.61	26.69

Sample #	14-3-1
Ht. of Sample	6.0
Tare	14
Gross Wet Wt	921.5
Gross Dry Wt.	816.6
Tare Wt.	109.6
Net Dry Wt.	707.0
Wt. Of Water	104.9
% Moisture	14.8%
Dry Density	101.3

Brown Silty Sand with Clay

Group Symbol	**SC**

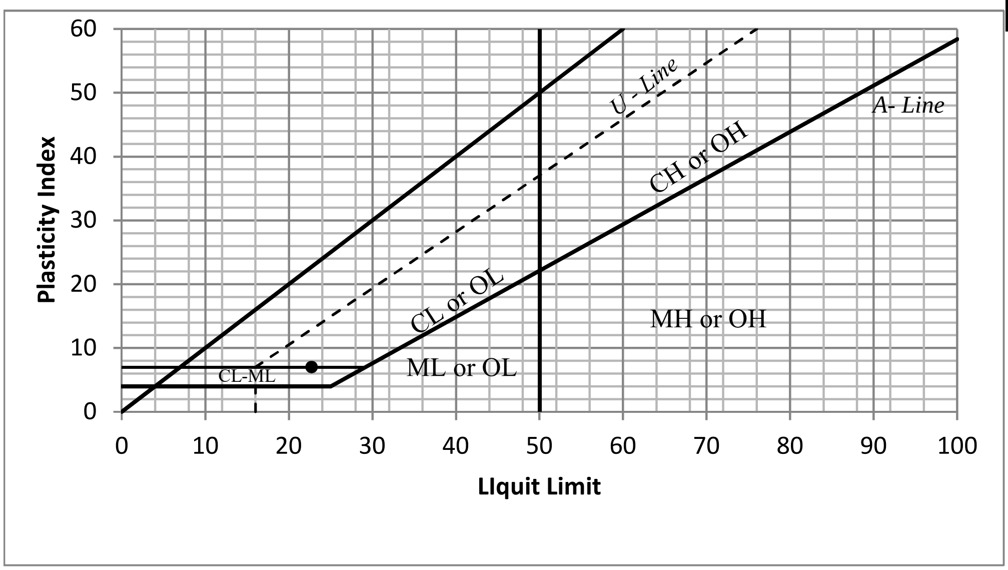


FIGURE NO. 37

Liquid Limit:	24.1
Plastic Limit:	15.4
Plasticity Index:	8.7

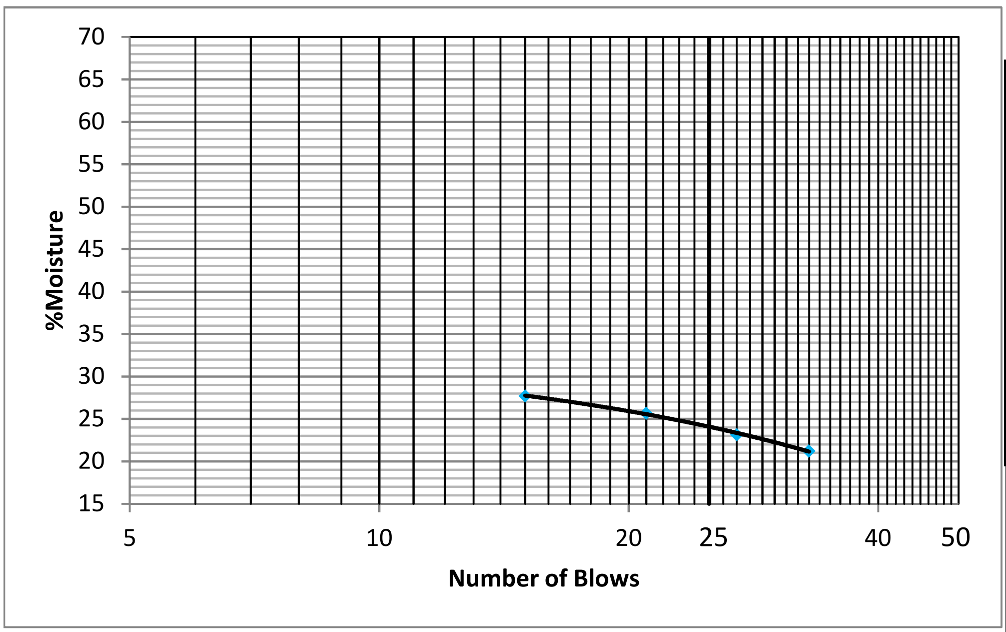
PI
9

HARO, KASUNICH AND ASSOCIATES, INC.
CONSULTING GEOTECHNICAL & COASTAL ENGINEERS

File N◦	SC 4090
Sample N◦	16-4
Date:	2/1/2018
By:	RC

P.I. SOIL TEST

	PLASTIC LIMIT				LIQUID LIMIT NUMBER OF BLOWS			
Determination	1	2	3	4	33	27	21	15
Tare N◦	26	31	22		4e	5b	3e	5g
Gross Wet WT.	14.92	16.02	17.48		14.24	12.21	16.41	16.15
GrossDry WT.	14.39	15.36	16.64		12.48	10.70	13.93	13.55
Tare WT.	11.00	10.98	11.19		4.19	4.18	4.28	4.17
NET DRY WT.	3.39	4.38	5.45	0.00	8.29	6.52	9.65	9.38
WT. OF Water	0.53	0.66	0.84	0.00	1.76	1.51	2.48	2.60
% Moisture	15.63	15.07	15.41	#DIV/0!	21.23	23.16	25.70	27.72



Sample #	
Ht. of Sample	bag
Tare	200
Gross Wet Wt	808.3
Gross Dry Wt.	725.9
Tare Wt.	81.2
Net Dry Wt.	644.7
Wt. Of Water	82.4
% Moisture	12.8%
Dry Density	#VALUE!

Description:
olive brown
Sandy lean Clay

Group Symbol	**SC**

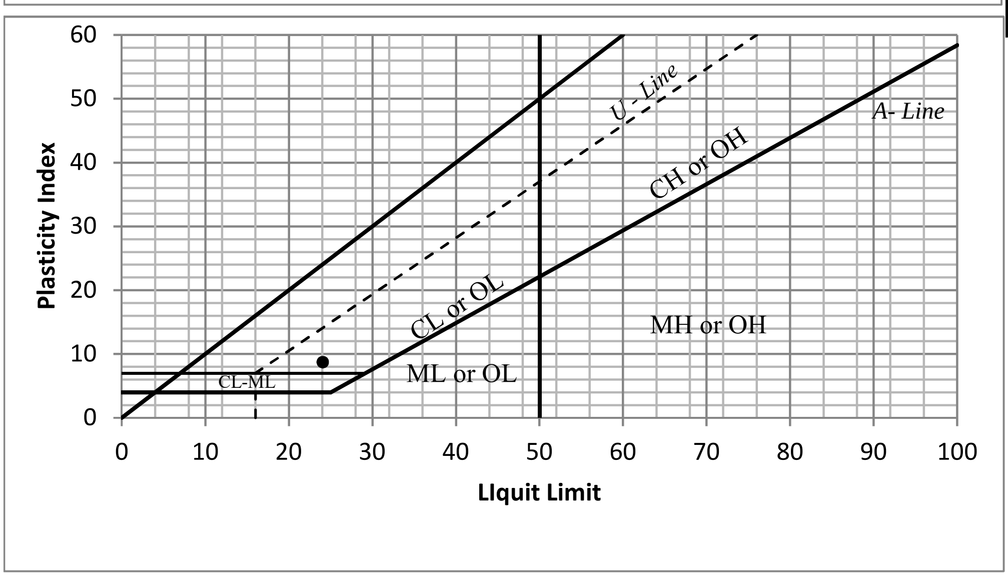


FIGURE NO. 38

Liquid Limit:	24.2
Plastic Limit:	19.4
Plasticity Index:	4.8

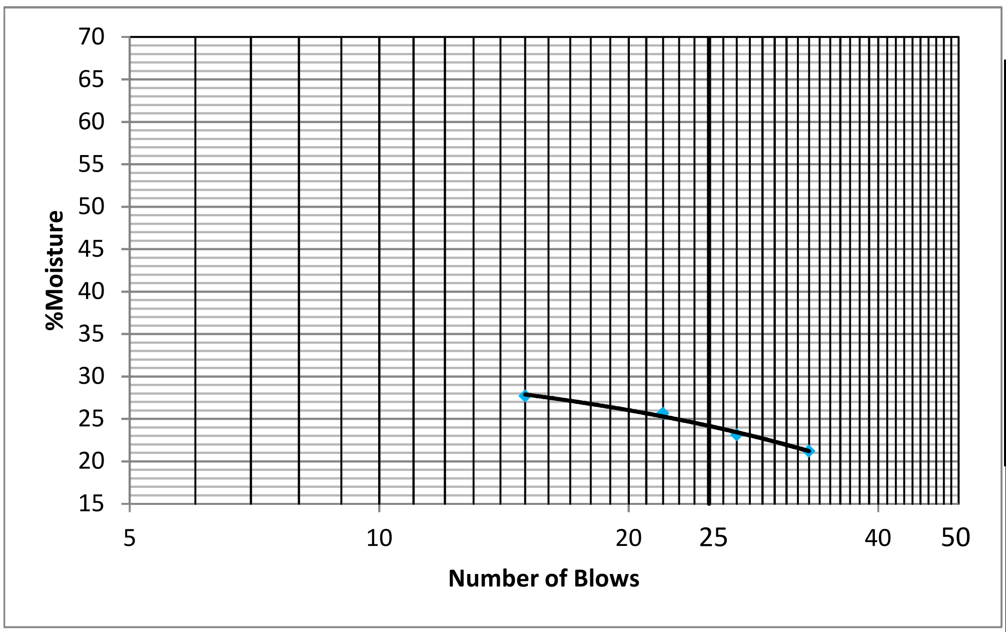
PI
5

HARO, KASUNICH AND ASSOCIATES, INC.
CONSULTING GEOTECHNICAL & COASTAL ENGINEERS

File N◦	SC 4090
Sample N◦	16-7
Date:	2/1/2018
By:	RC

P.I. SOIL TEST

	PLASTIC LIMIT				LIQUID LIMIT NUMBER OF BLOWS			
Determination	1	2	3	4	33	27	22	15
Tare N◦	16	12	27		4e	5b	3e	5g
Gross Wet WT.	13.71	14.87	15.49		14.24	12.21	16.41	16.15
GrossDry WT.	13.28	14.22	14.78		12.48	10.70	13.93	13.55
Tare WT.	10.99	10.96	11.09		4.19	4.18	4.28	4.17
NET DRY WT.	2.29	3.26	3.69	0.00	8.29	6.52	9.65	9.38
WT. OF Water	0.43	0.65	0.71	0.00	1.76	1.51	2.48	2.60
% Moisture	18.78	19.94	19.24	#DIV/0!	21.23	23.16	25.70	27.72



Sample #	
Ht. of Sample	bag
Tare	11
Gross Wet Wt	367.6
Gross Dry Wt.	339.8
Tare Wt.	110.4
Net Dry Wt.	229.4
Wt. Of Water	27.8
% Moisture	12.1%
Dry Density	#VALUE!

Description:	
Dark Grey	
Sandy lean Clay	
Group Symbol	**CL-ML**

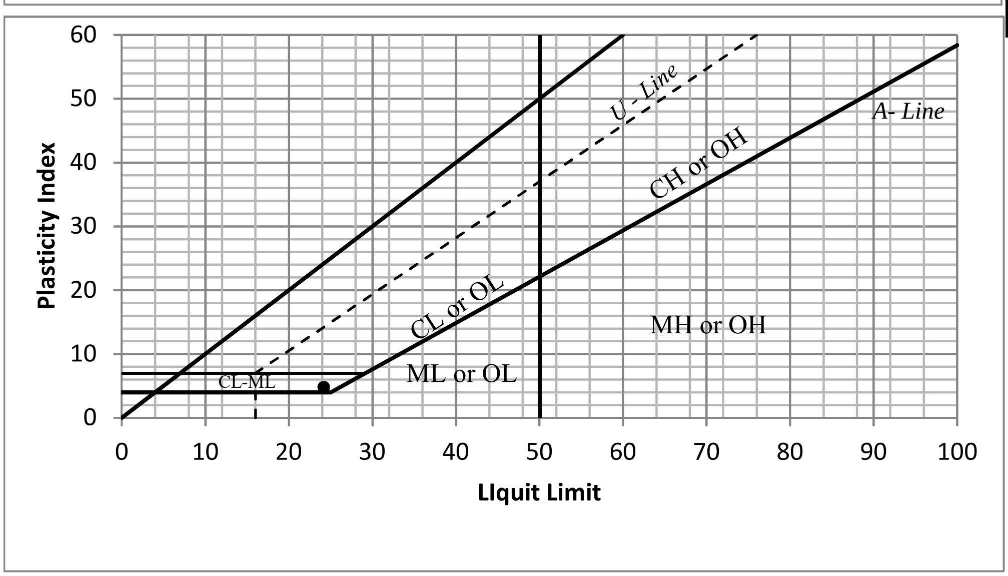


FIGURE NO. 39

Direct Shear

Project:	SLVWD LYON TANK	**Date**	1/5/2018
Date	1/5/2018	**Tested By:**	RC
Description	Gold/Light Brown D.G. w/ Clay		

Test Number	1	2	3	4
Normal Pressure (PSF)	530	1030	2030	4030
Max Shear Stress	17.7	32.4	55.3	0
Shear Stress (PSF)	521.3	953.3	1627.2	-

Equation of Trendline	
Intercept	Slope
162.473	0.7282

*Manually Enter from Trendline Equation

C (PSF)	PHI
162	36

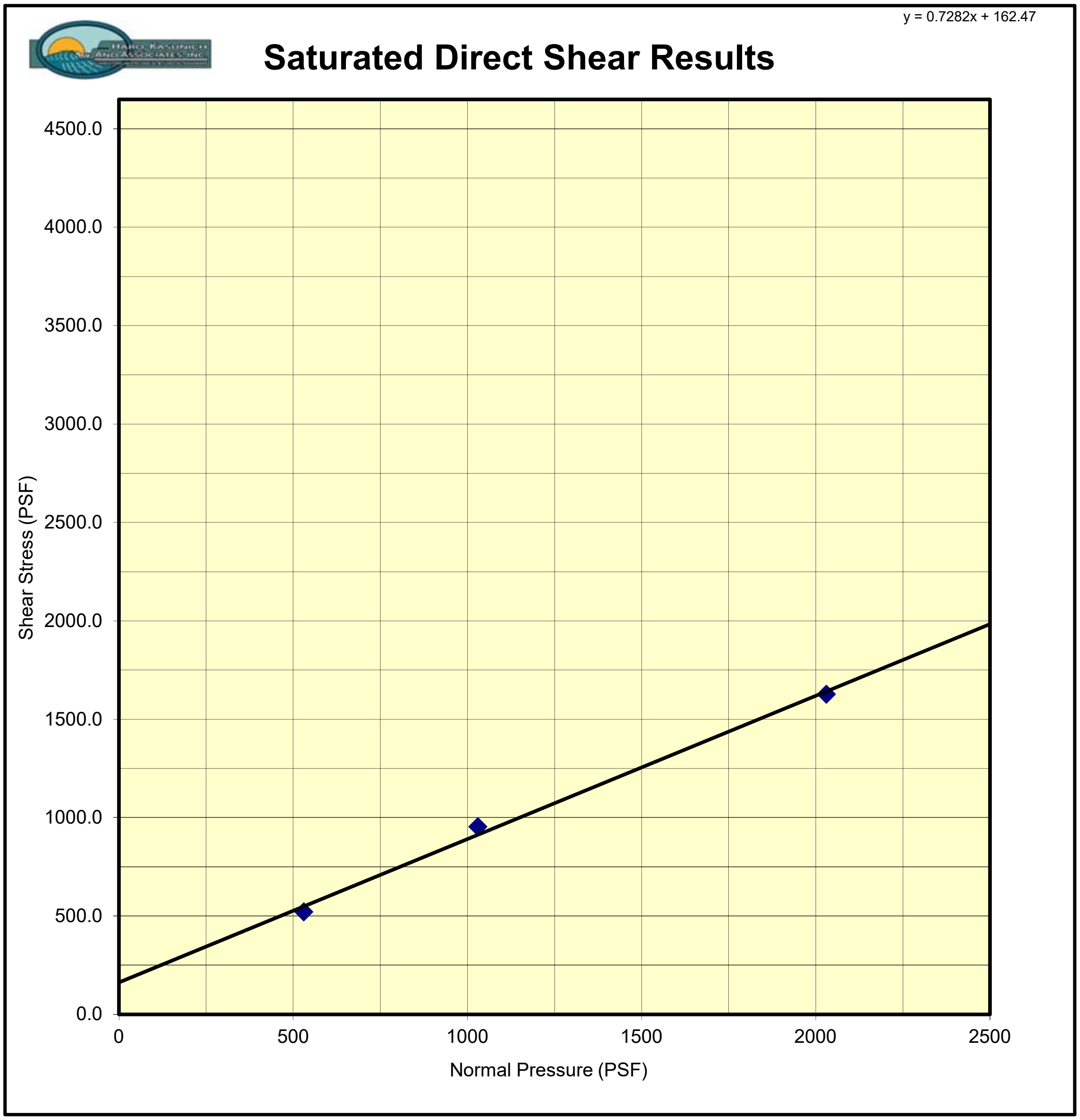


FIGURE NO. 40

Direct Shear

Project:	SLVWD LYON TANK		**Date**	1/5/2018
Sample #	5-7-1		**Tested By:**	RC
Description	Brown Silty Sand and D.G.			

Test Number	1	2	3	4
Normal Pressure (PSF)	530	1030	2030	4030
Max Shear Stress	29.3	52.5	92.7	0
Shear Stress (PSF)	861.1	1543.7	2727.4	-

Equation of Trendline	
Intercept	Slope
232.21	1.2355

*Manually Enter from Trendline Equation

C (PSF)	PHI
232	51

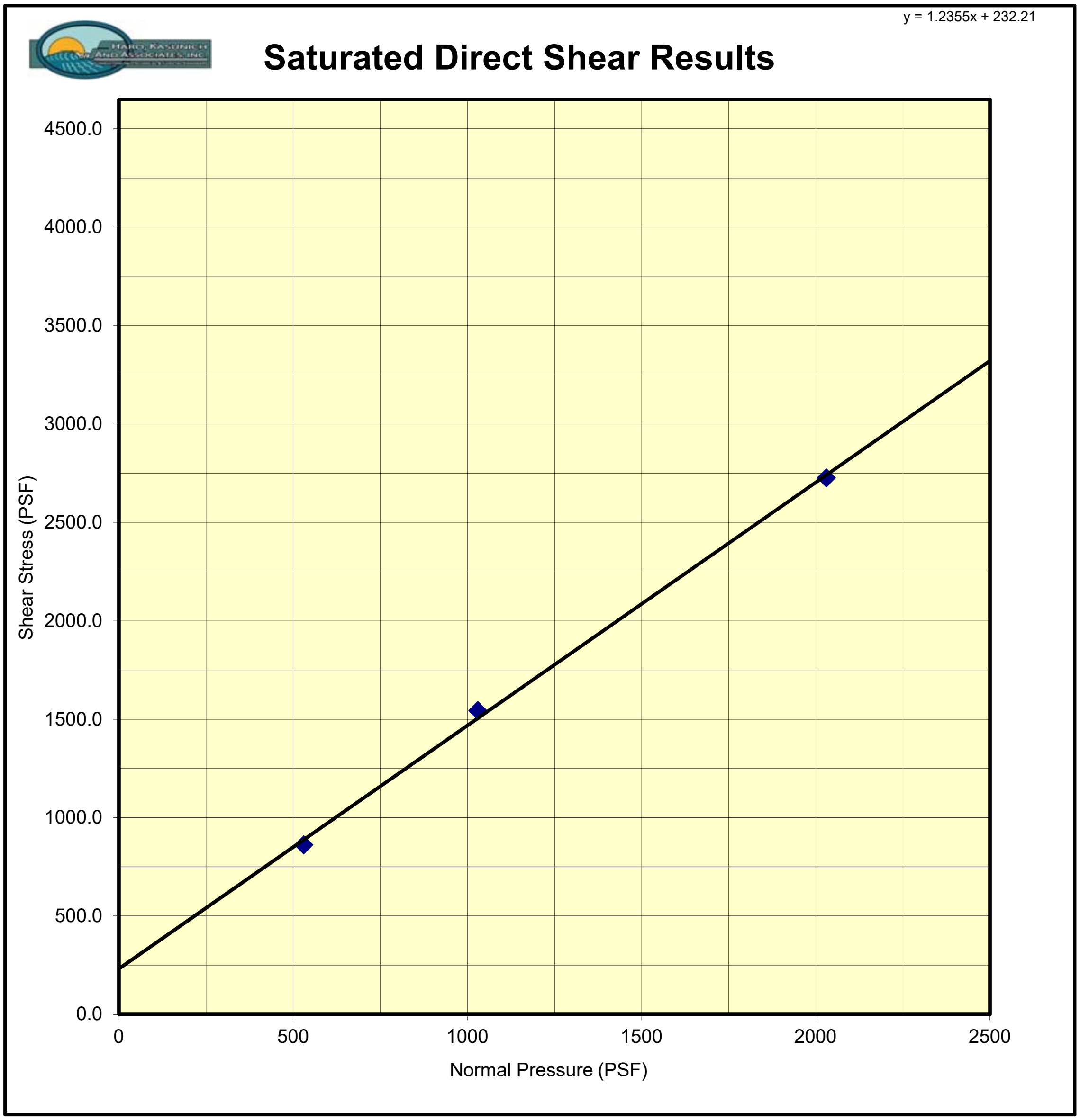


FIGURE NO. 41

Direct Shear

Project:	SLVWD LYON TANK		**Date**	1/5/2018
Sample #	6-3-1		**Tested By:**	RC
Description	Brown Silty Sand and D.G.			

Test Number	1	2	3	4
Normal Pressure (PSF)	530	1030	2030	4030
Max Shear Stress	32.7	57.4	89.1	0
Shear Stress (PSF)	961.9	1687.7	2620.8	-

Equation of Trendline	
Intercept	Slope
462.9	1.0812

*Manually Enter from Trendline Equation

C (PSF)	PHI
463	47

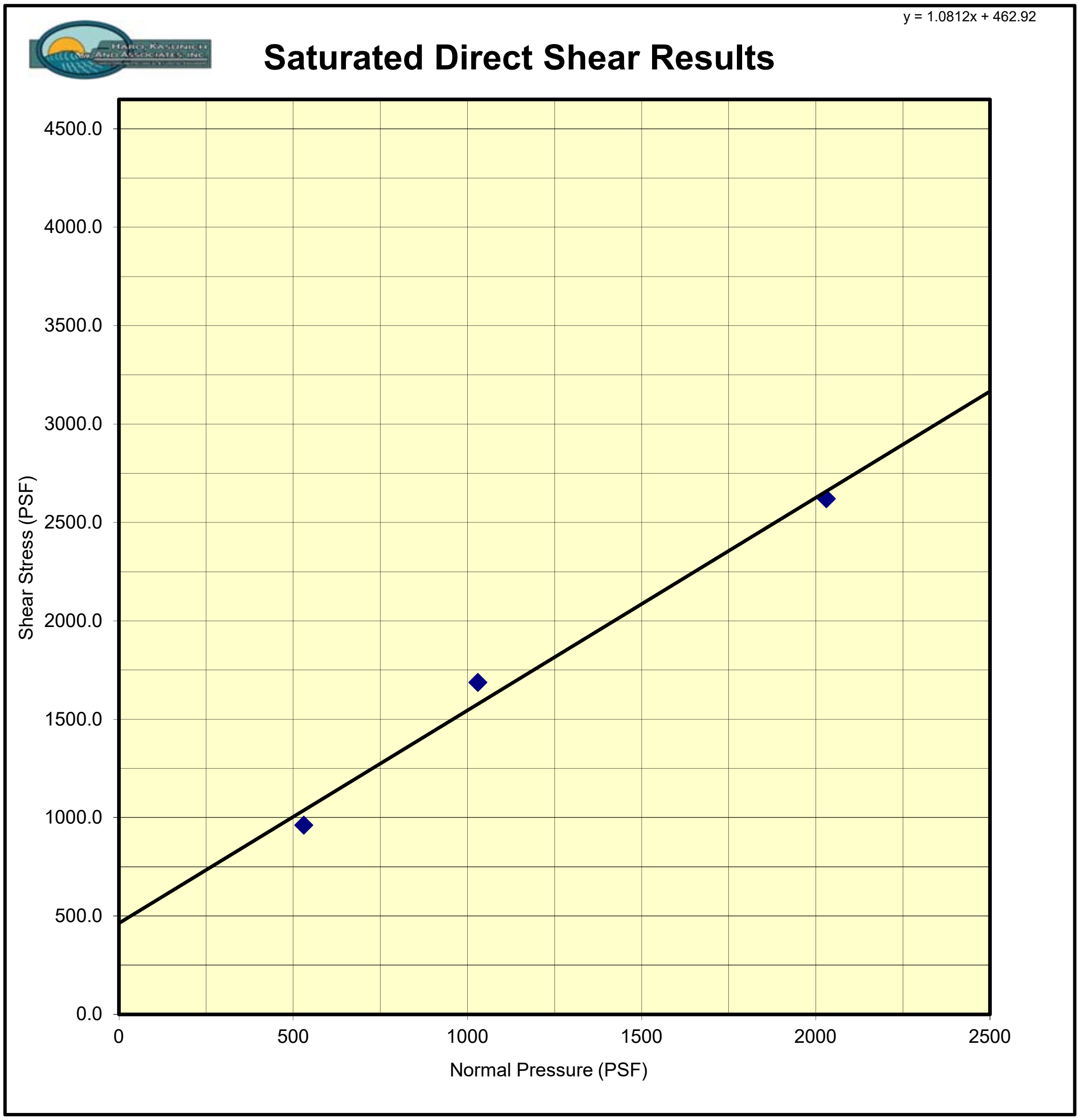


FIGURE NO. 42

Direct Shear

Project:	SLVWD LYON TANK
Sample #	6-9-1
Description	Brown Sandy Clay

Date	1/5/2018
Tested By:	RC/MM

Test Number	1	2	3	4
Normal Pressure (PSF)	530	1030	2030	4030
Max Shear Stress	31.7	50.4	70.8	0
Shear Stress (PSF)	933.1	1481.8	2082.2	0

Equation of Trendline	
Intercept	Slope
610.61	0.7424

C (PSF)	PHI
611	37

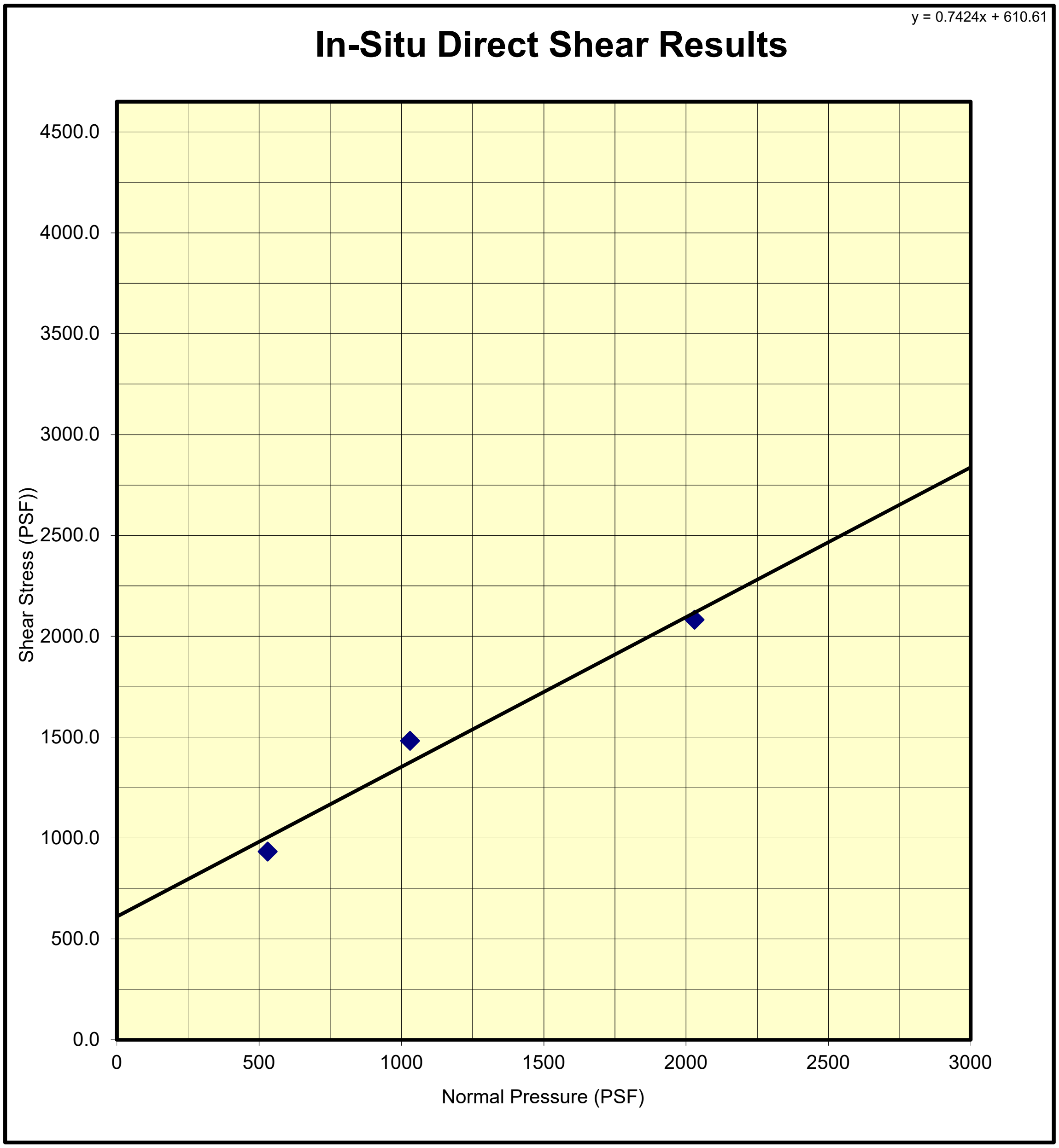


FIGURE NO. 43

Direct Shear

Project:	SLVWD LYON TANK		**Date**	1/5/2018
Sample #	1/5/2018		**Tested By:**	RC
Description	Mottled Brown Silty Sand			

Test Number	1	2	3	4
Normal Pressure (PSF)	530	1030	2030	4030
Max Shear Stress	34.2	36.0	78.1	0
Shear Stress (PSF)	1005.1	1058.4	2296.8	-

Equation of Trendline	
Intercept	Slope
358.47	0.915

*Manually Enter from Trendline Equation

C (PSF)	PHI
358	42

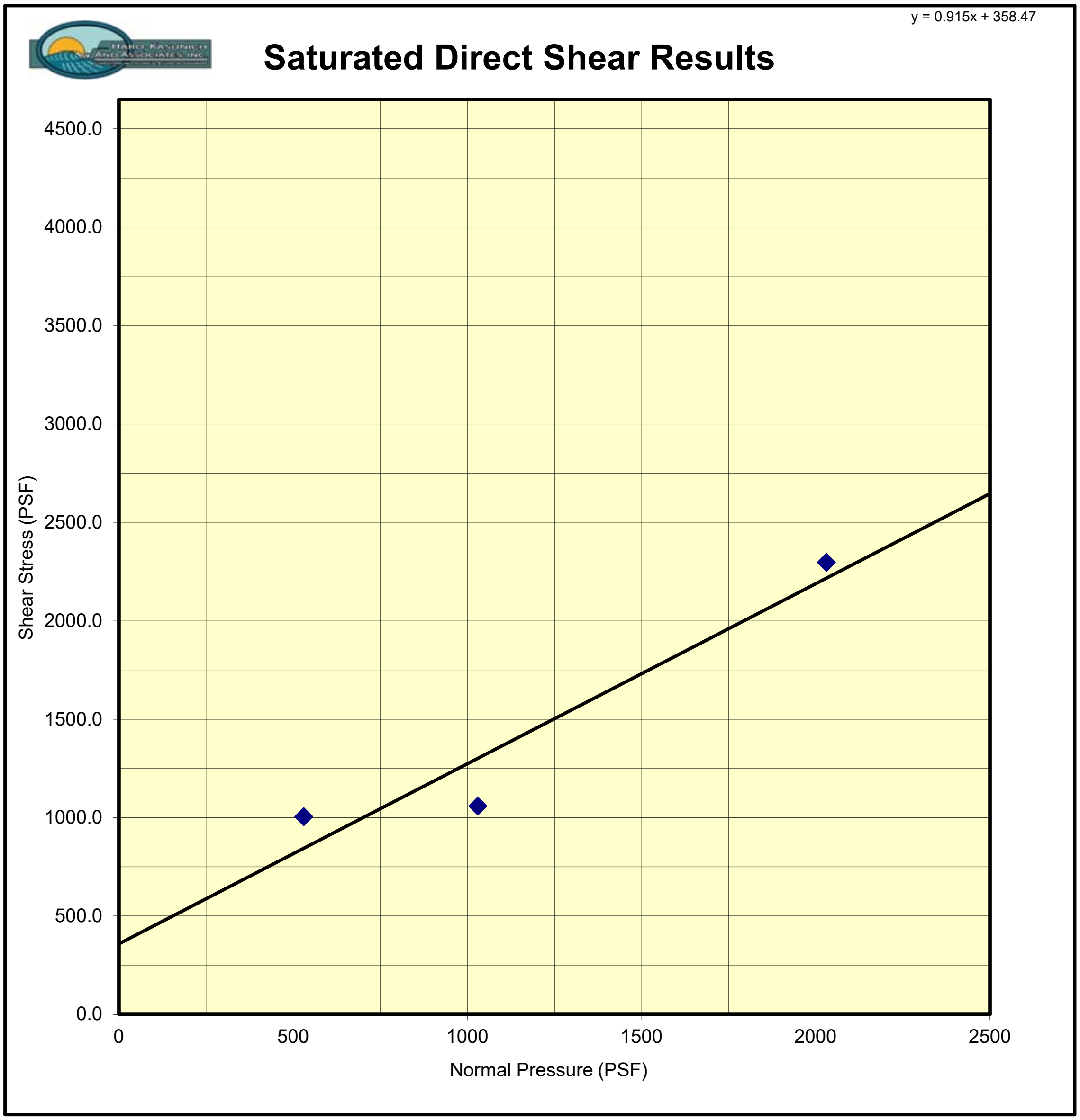


FIGURE NO. 44

Direct Shear

Project:	SLVWD LYON TANK
Sample #	8-6-2
Description	Brown Clayey Sand

Date	12/15/2017
Tested By:	RC/MM

Test Number	1	2	3	4
Normal Pressure (PSF)	530	1030	2030	4030
Max Shear Stress	18.5	25.8	47.8	117.8467
Shear Stress (PSF)	544.3	758.9	1405.4	3466.08

Equation of Trendline	
Intercept	Slope
0	0.8244

C (PSF)	PHI
0	40

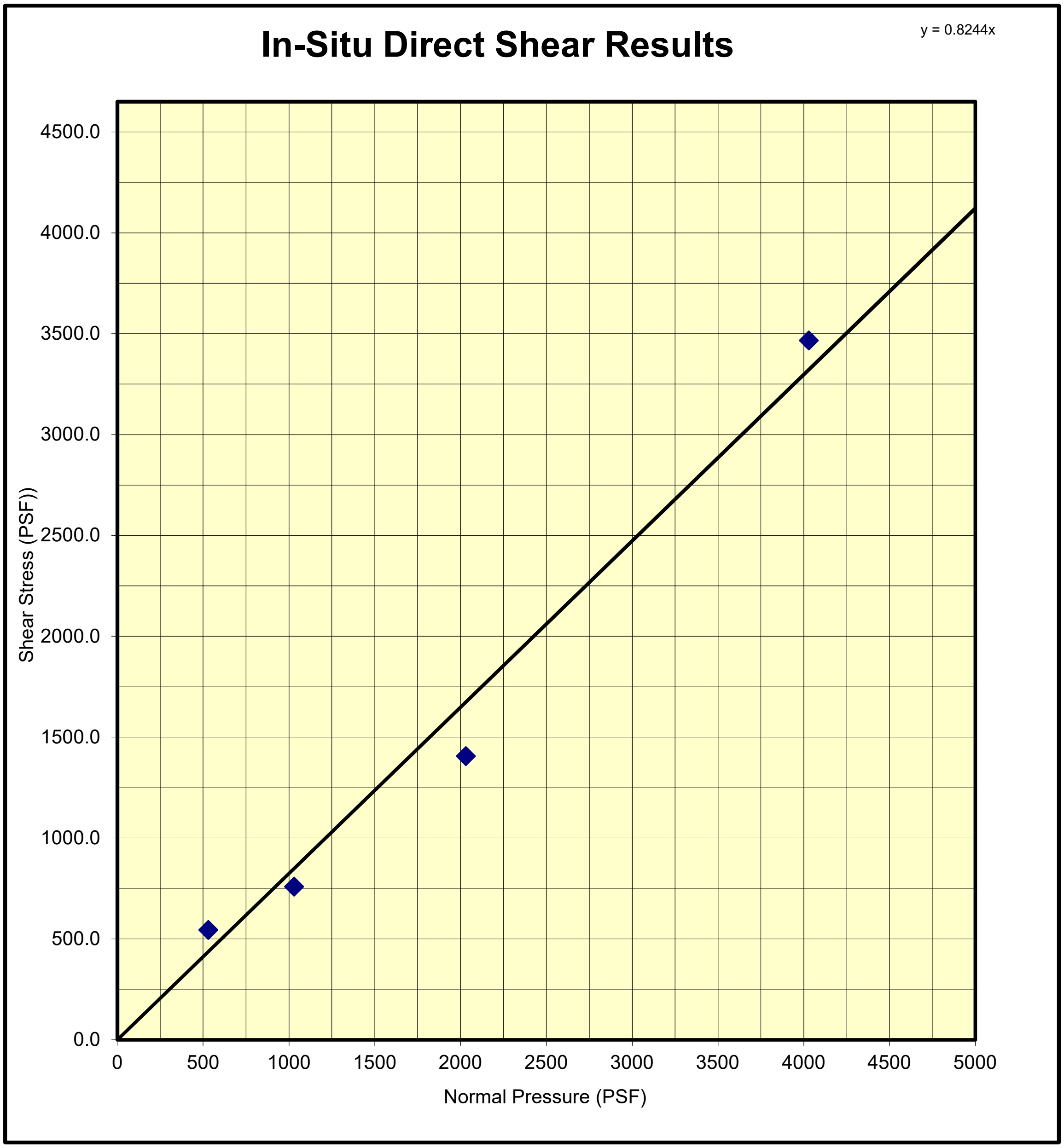


FIGURE NO. 45

Direct Shear

Project:	SLVWD Lyon Tank AR Slide
Sample #	11-7-2
Description	Brown Silty Sand

Date	1/5/2018
Tested By:	RC

Test Number	1	2	3	4
Normal Pressure (PSF)	530	1030	2030	4030
Max Shear Stress	27.8	27.8	57.1	0
Shear Stress (PSF)	816.5	816.5	1679.0	-

Equation of Trendline	
Intercept	Slope
366.72	0.6161

*Manually Enter from Trendline Equation

C (PSF)	PHI
367	32

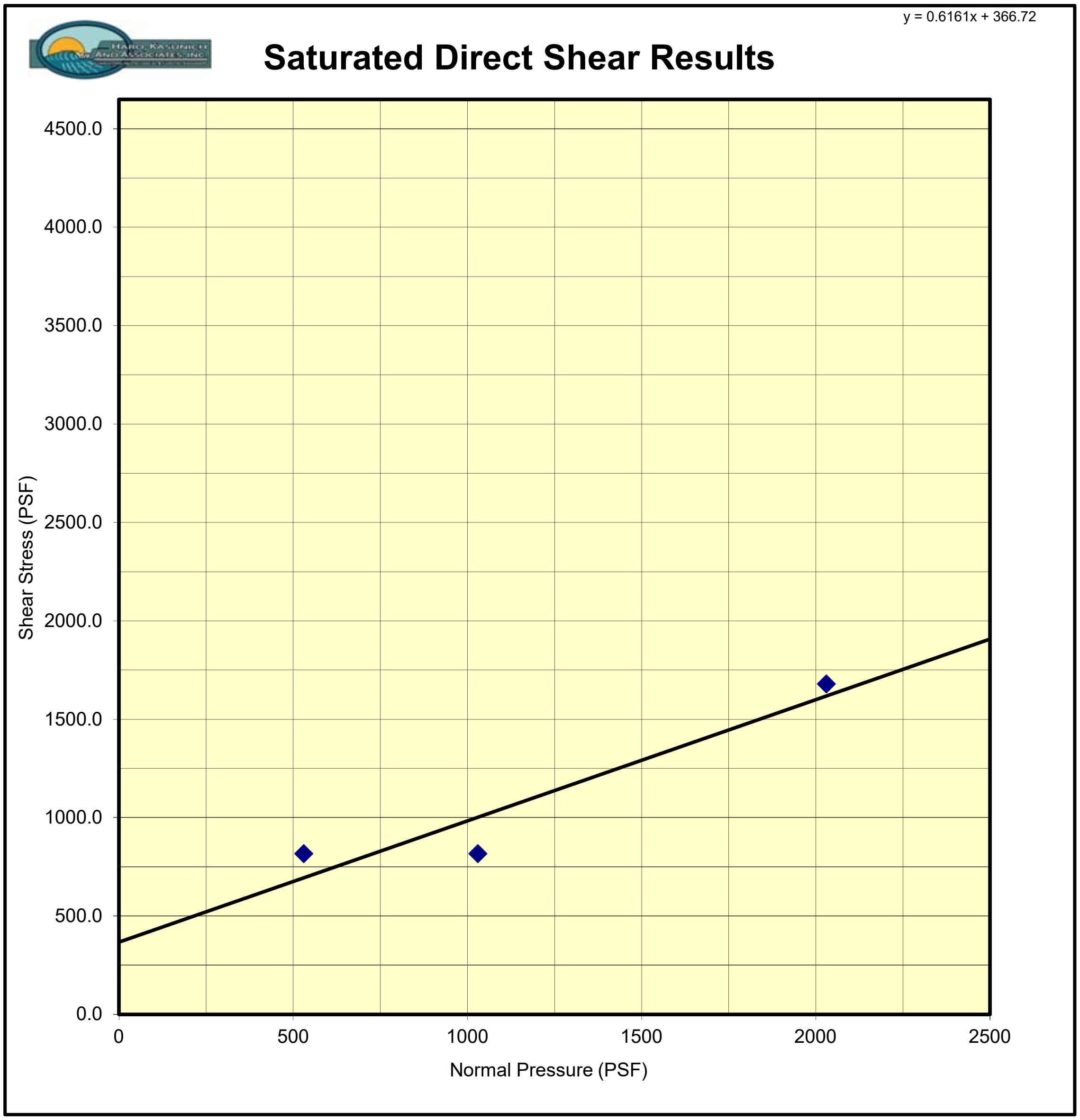


FIGURE NO. 46

Direct Shear

Project:	SLVWD LYON TANK
Sample #	12-8-2
Description	Orangish brown clay w/ sand

Date	1/5/2018
Tested By:	RC

Test Number	1	2	3	4
Normal Pressure (PSF)	530	1030	2030	4030
Max Shear Stress	28.1	45.1	79.3	0
Shear Stress (PSF)	825.1	1327.7	2332.8	-

Equation of Trendline	
Intercept	Slope
292.41	1.0051

*Manually Enter from Trendline Equation

C (PSF)	PHI
292	45

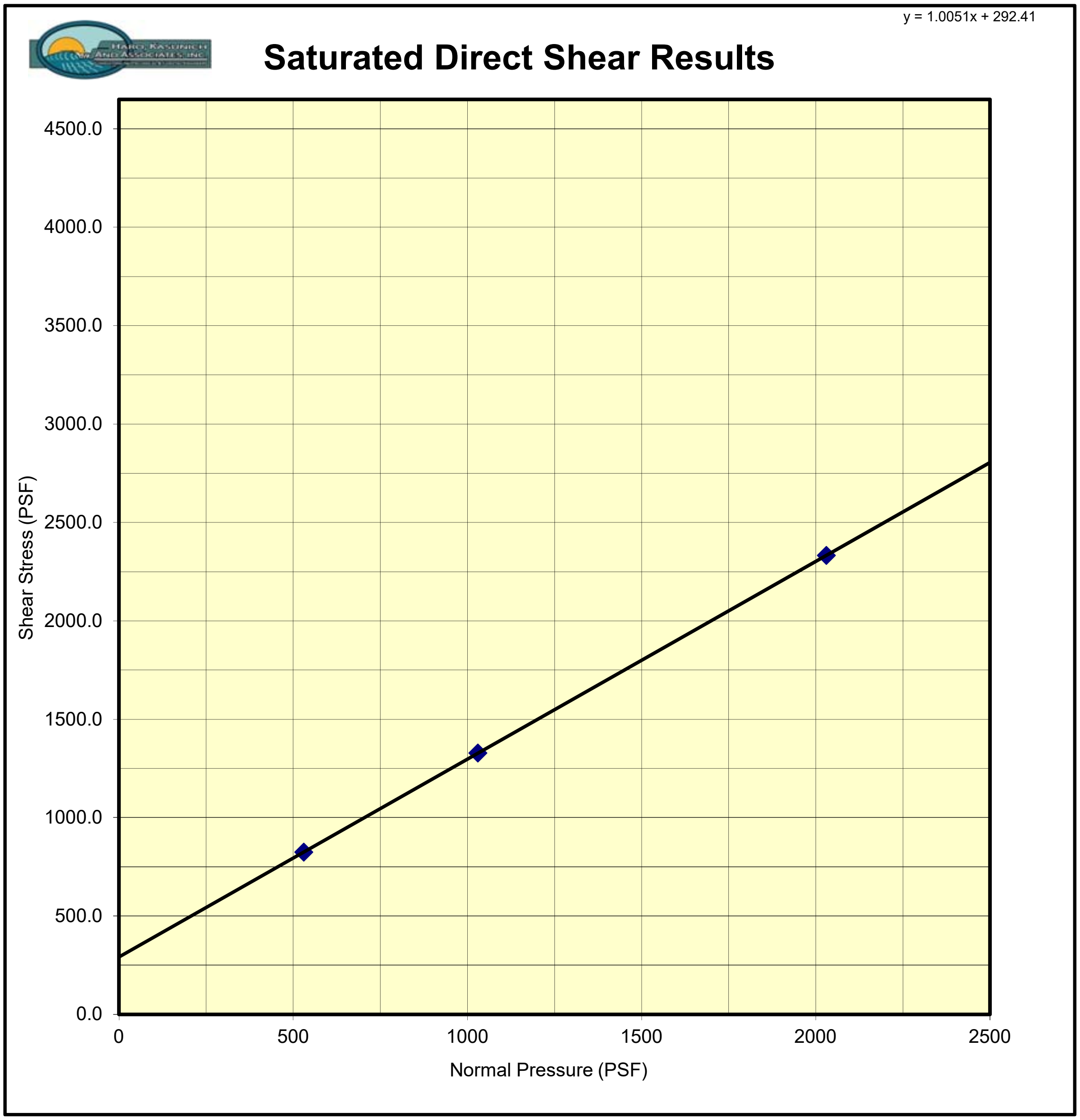
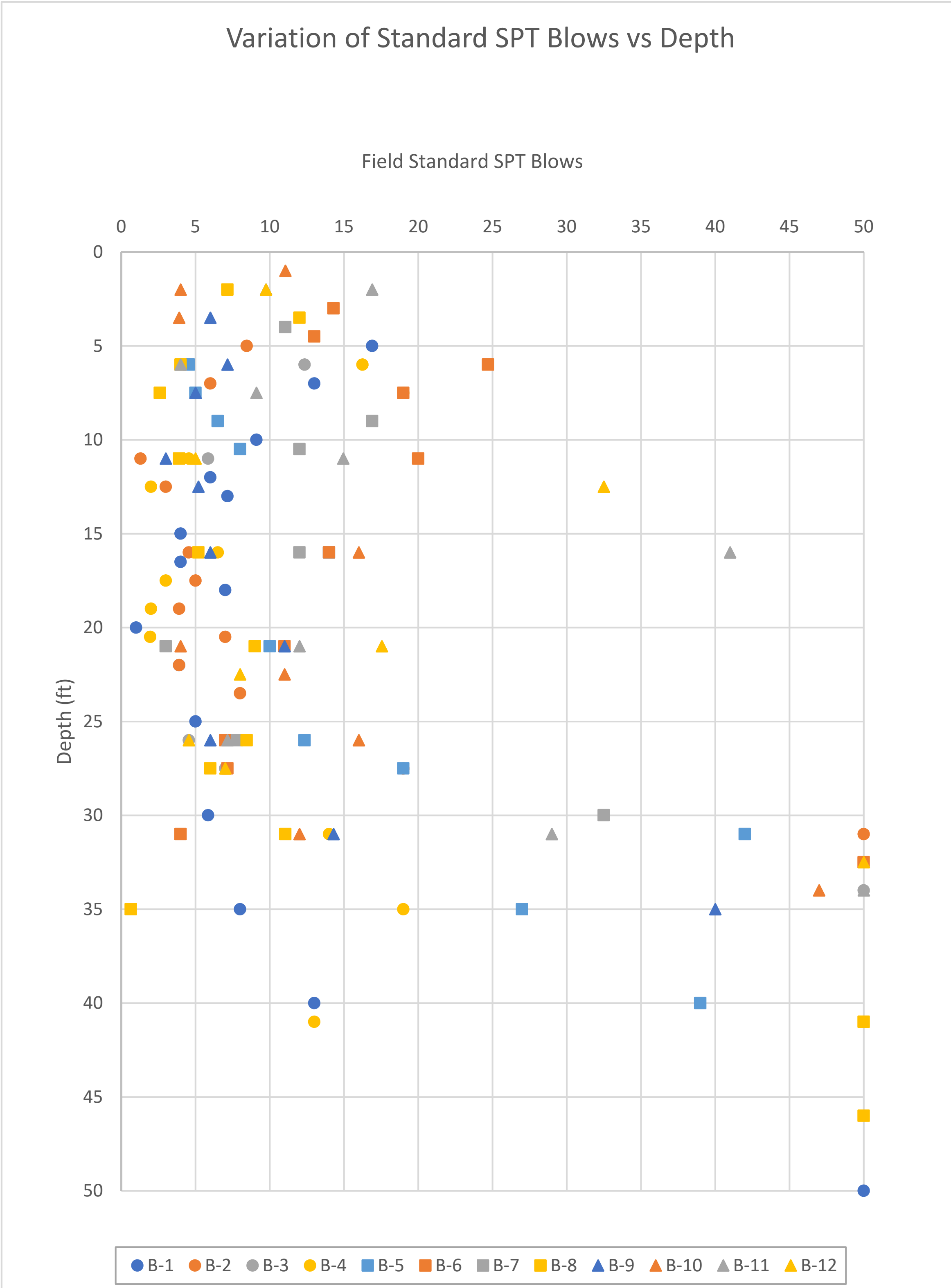


FIGURE NO. 47



Variation of Standard SPT Blows vs Depth
Field Standard SPT Blows
0
5
10
15
20
25
30
35
40
45
50
Depth (ft)
B-1
B-2
B-3
B-4
B-5
B-6
B-7
B-8
B-9
B-10
B-11
B-12

FIGURE NO. 48

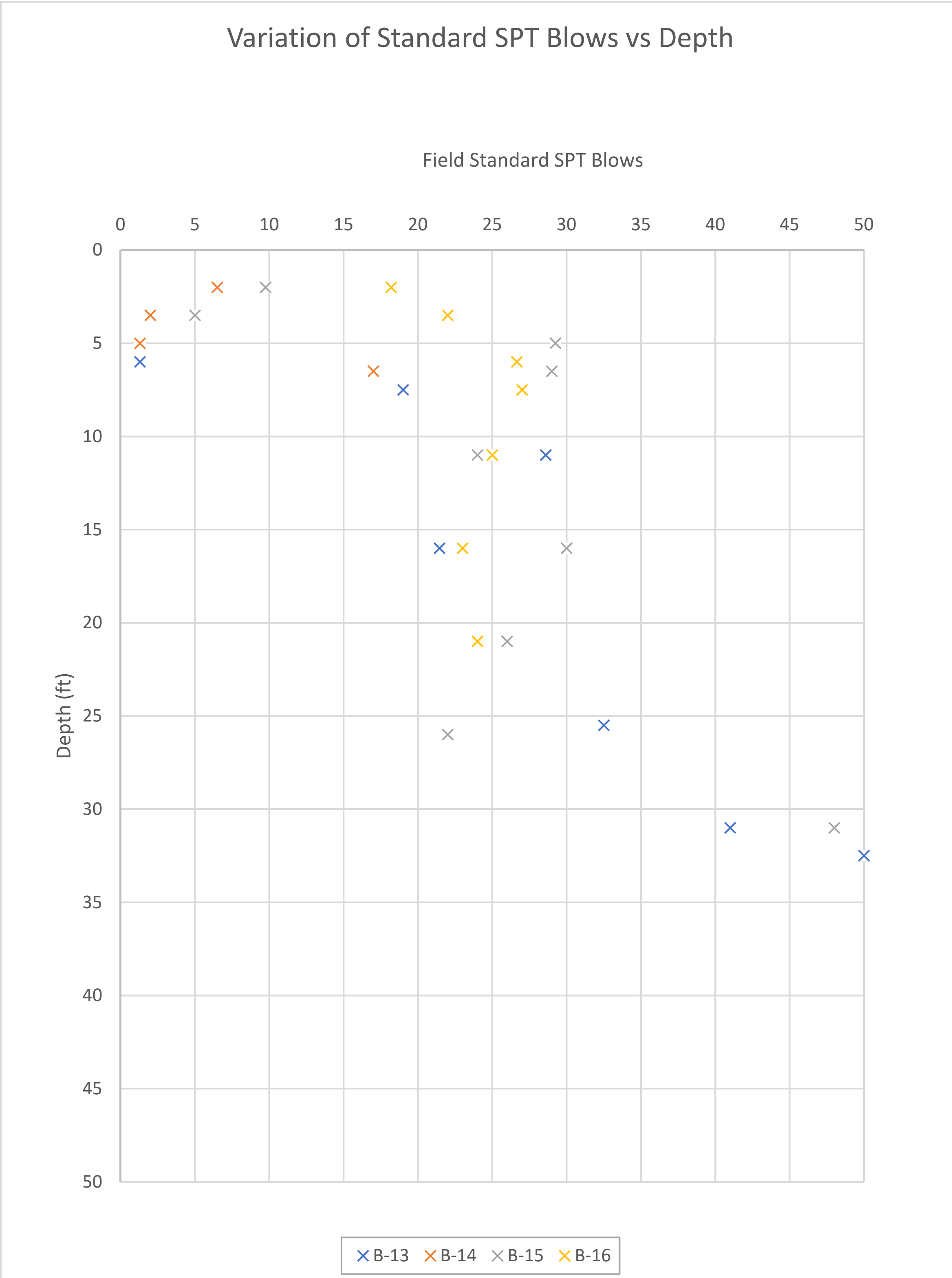
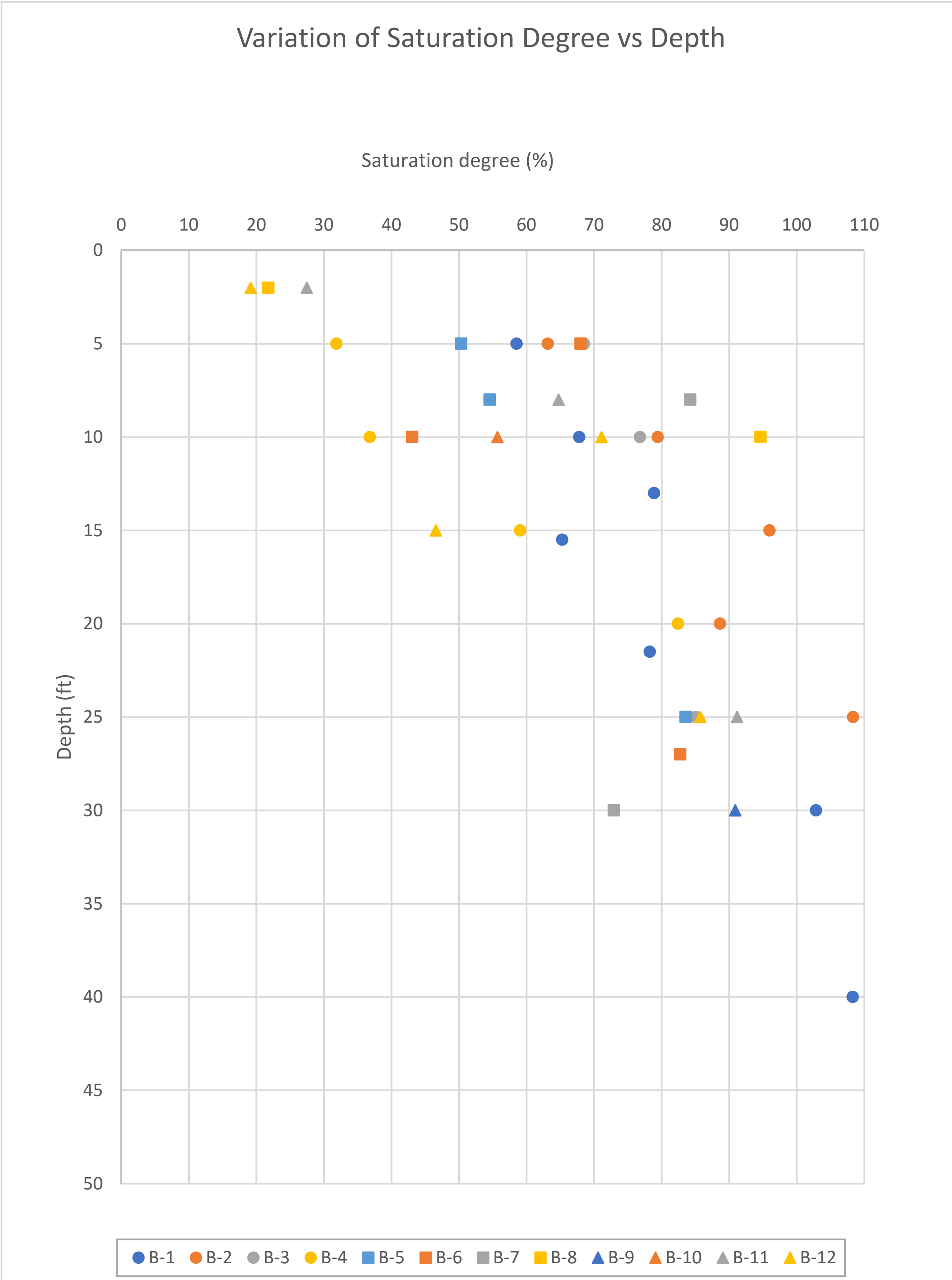

Variation of Standard SPT Blows vs Depth
Field Standard SPT Blows
0
5
10
15
20
25
30
35
40
45
50
Depth (ft)
B-13
B-14
B-15
B-16

FIGURE NO. 49



Variation of Saturation Degree vs Depth
Saturation degree (%)
0
10
20
30
40
50
60
70
80
90
100
110
0
5
10
15
20
25
30
35
40
45
50
Depth (ft)
B-1
B-2
B-3
B-4
B-5
B-6
B-7
B-8
B-9
B-10
B-11
B-12

FIGURE NO. 50

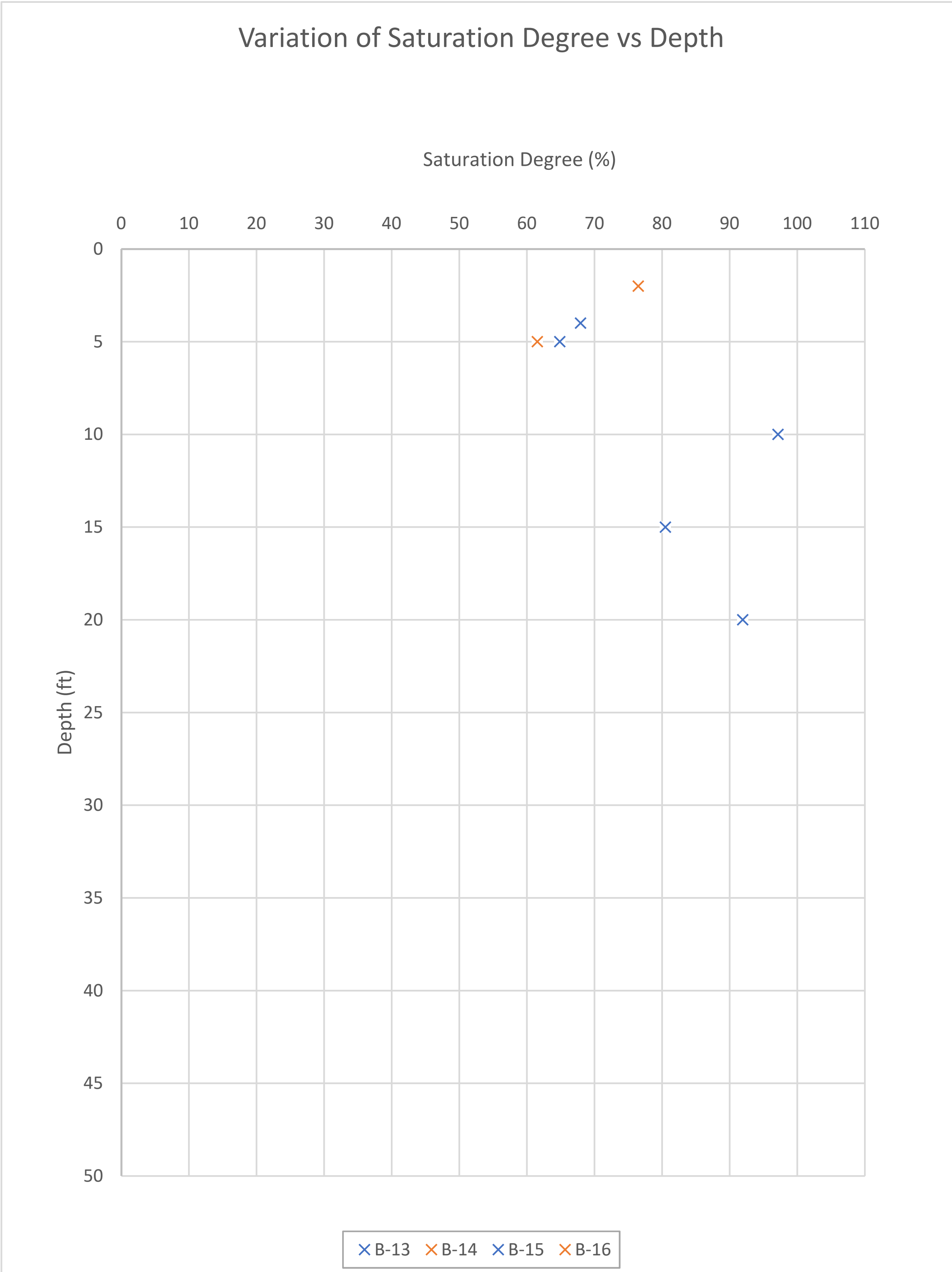

Variation of Saturation Degree vs Depth
Saturation Degree (%)
0
10
20
30
40
50
60
70
80
90
100
110
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
B-13
B-14
B-15
B-16

FIGURE NO. 51

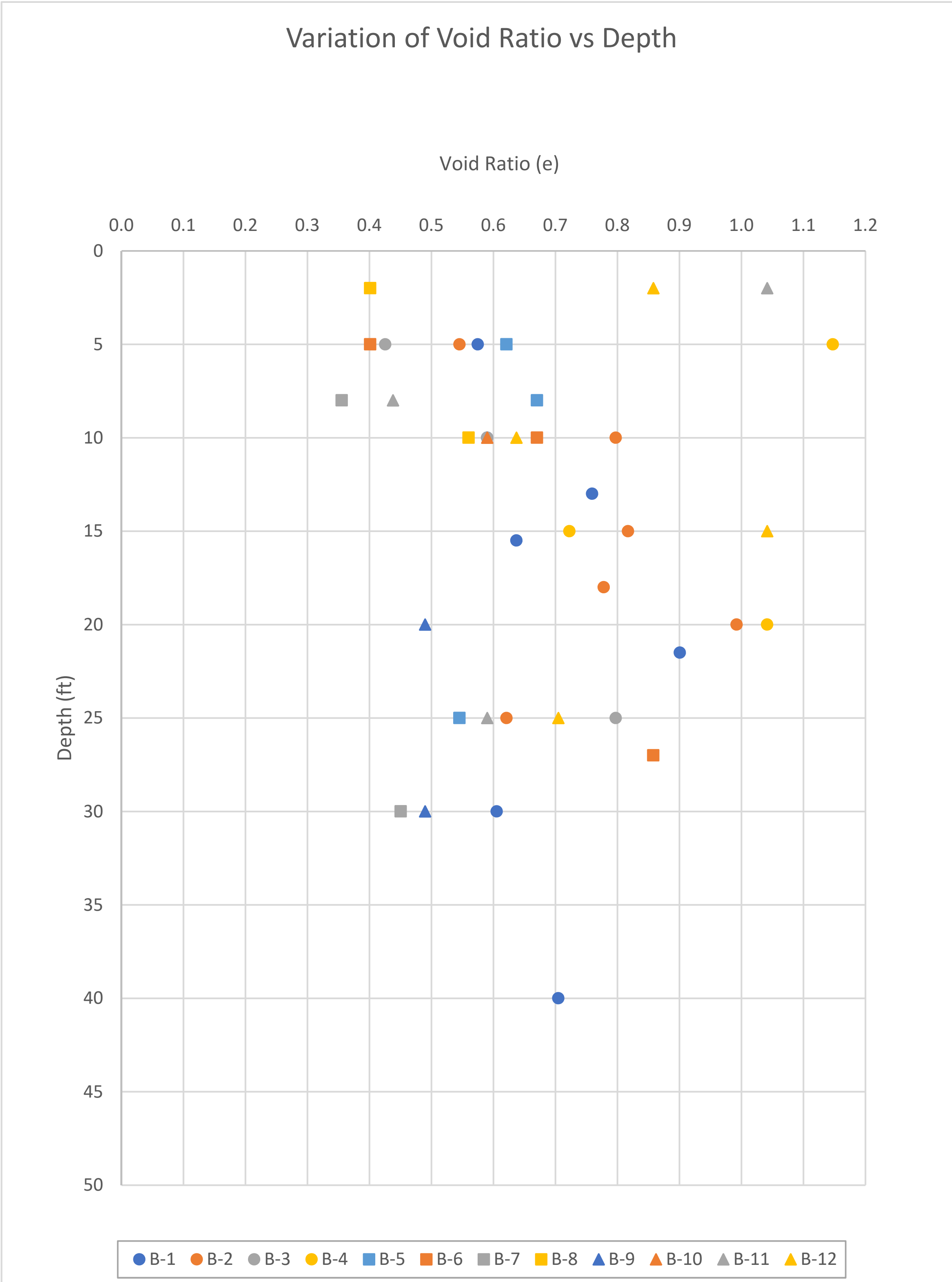
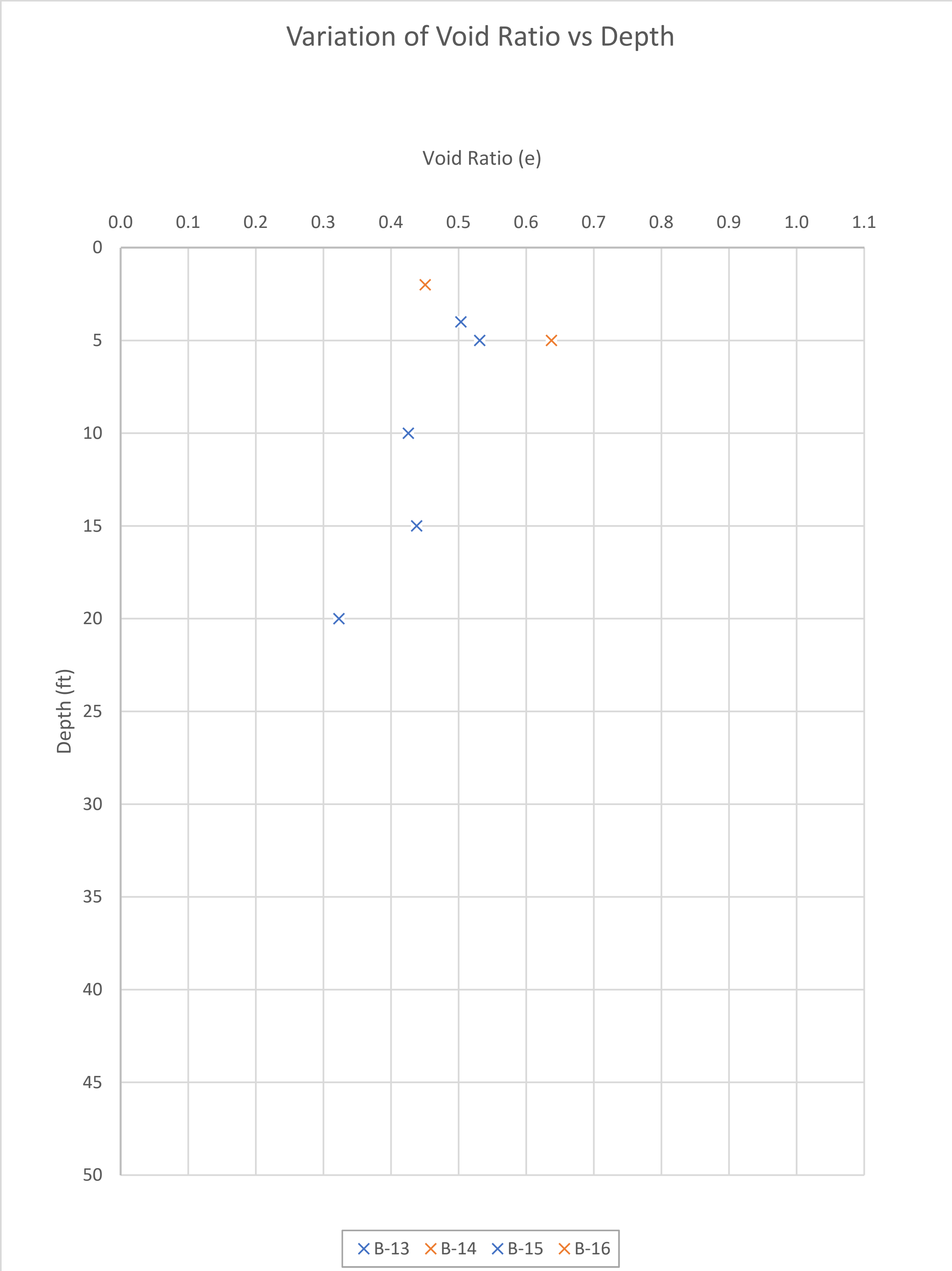

Variation of Void Ratio vs Depth
Void Ratio (e)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.1
1.2
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
B-1
B-2
B-3
B-4
B-5
B-6
B-7
B-8
B-9
B-10
B-11
B-12

FIGURE NO. 52



Variation of Void Ratio vs Depth
Void Ratio (e)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.1
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
B-13
B-14
B-15
B-16

FIGURE NO. 53

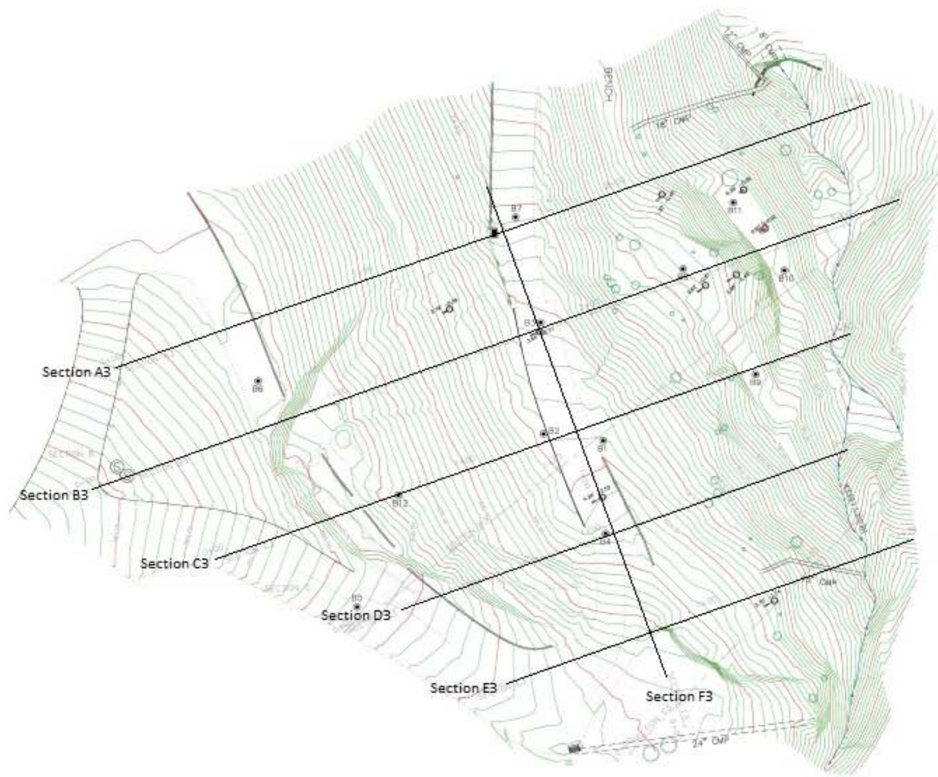
Project No. SC4090.1
6 August 2018

APPENDIX B

Lyon Tank Slide 3D Orthographical Model (Figures 54 – 58)

LEGEND:

= EXPOSED FISSURE OCTOBER 17, 2017
= EDGE OF PAVEMENT
= SEPTIC MANHOLE
= BORE SITE
= MONITORING POINT
= MOVEMENT OF MONITORING POINT SINCE FEBRUARY 25, 2017

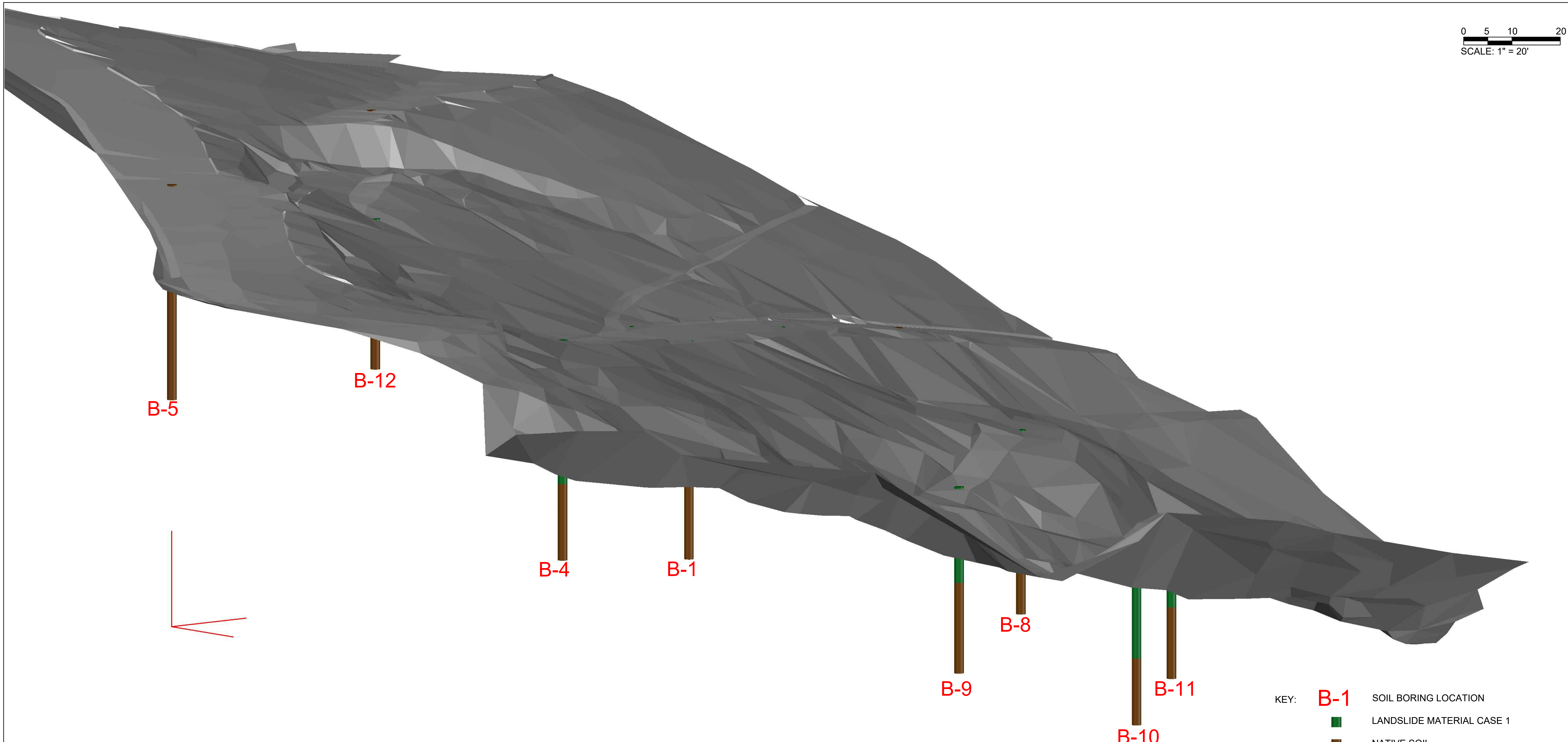


1" = 20'

TOPOGRAPHIC MAP

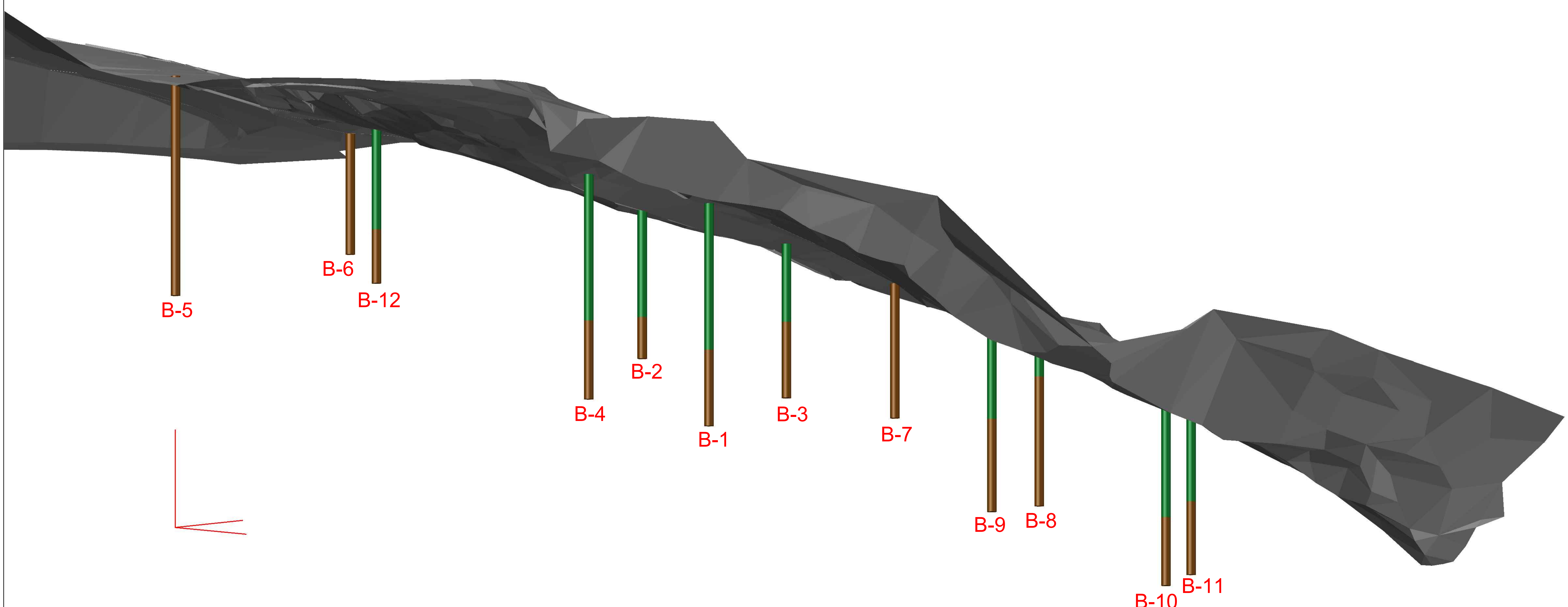
OF THE LANDS OF
SAN LORENZO VALLEY
WATER DISTRICT
LYON WATER TREATMENT PLANT
365 MADRONE DRIVE
BOULDER CREEK, CALIFORNIA
APN 081-011-07
OCTOBER, 2017

PREPARED BY
PAUL JENSEN
PROFESSIONAL LAND SURVEYOR * 4627
SANTA CRUZ, CALIFORNIA



NOTES:
1. 3D SURFACE FROM PAUL JENSEN PROFESSIONAL LAND SURVEYOR, DATED 2-1-2018.
2. BOTTOM OF BORING INDICATES BEDROCK CONTACT.

CASE 1 - ORTHOGRAPHIC VIEW LYONS TANK SLIDE SANTA CRUZ, CALIFORNIA SAN LORENZO VALLEY WATER DISTRICT	
SCALE: 1" = 20'	HARO, KASUNICH & ASSOCIATES, INC. GEOTECHNICAL AND COASTAL ENGINEERS 116 E. LAKE AVENUE, WATSONVILLE, CA 95076 (831) 722-4175
DRAWN BY: BRS	
DATE: MAY 2018	
REVISED:	
JOB NO. SC4090.1	
FIGURE NO. 55	SHEET NO.



NOTES:
1. 3D SURFACE FROM PAUL JENSEN PROFESSIONAL LAND SURVEYOR, DATED 2-1-2018.
2. BOTTOM OF BORING INDICATES BEDROCK CONTACT.

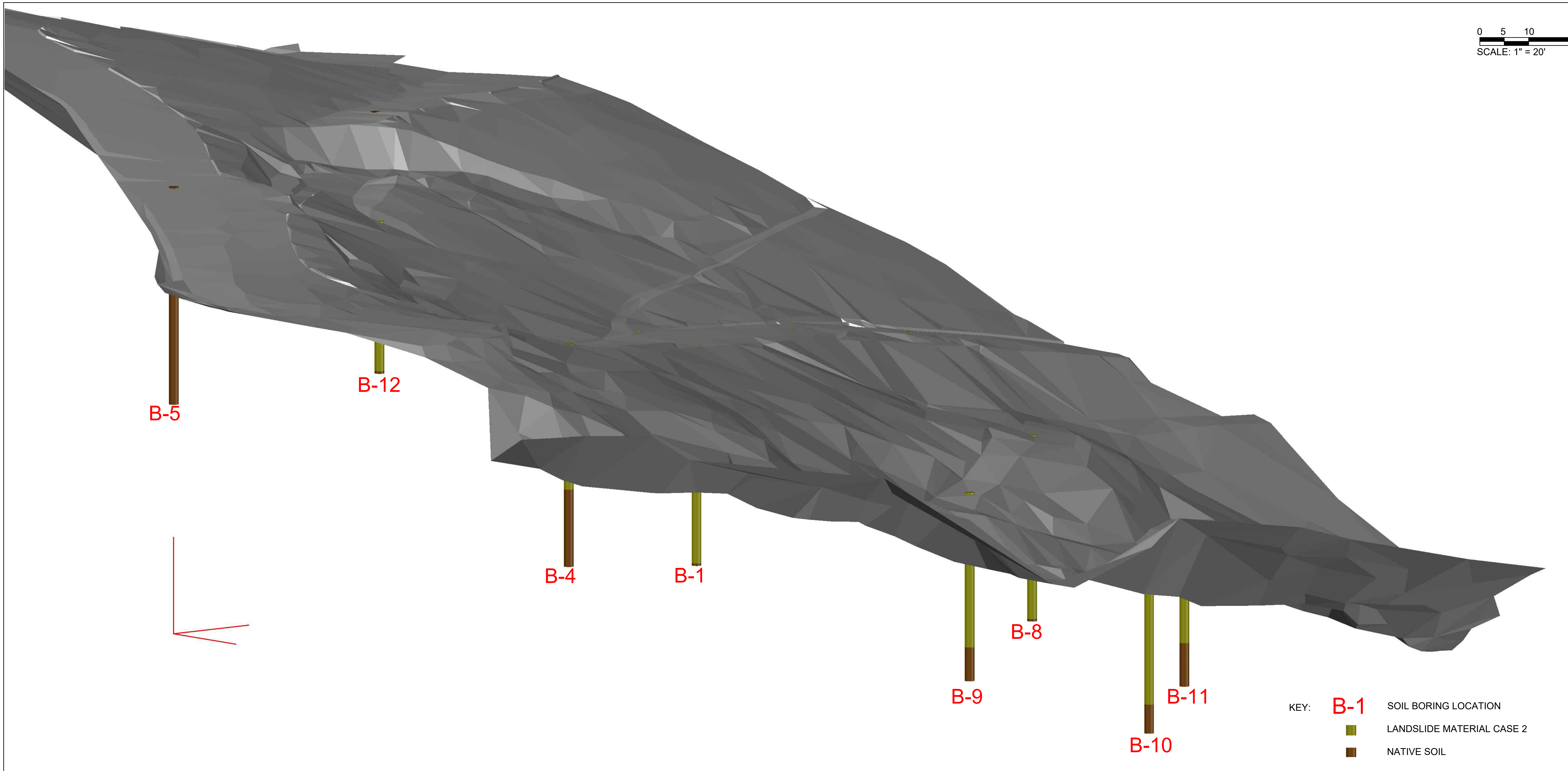
CASE 1 - ORTHOGRAPHIC VIEW
LYONS TANK SLIDE
SANTA CRUZ, CALIFORNIA
SAN LORENZO VALLEY WATER DISTRICT

SCALE:	1" = 20'
DRAWN BY:	BRS
DATE:	MAY 2018
REVISED:	
JOB NO.	SC4090.1

HARO, KASUNICH & ASSOCIATES, INC.
GEOTECHNICAL AND COASTAL ENGINEERS
116 E. LAKE AVENUE, WATSONVILLE, CA 95076
(831) 722-4175

FIGURE NO. 56

SHEET NO.



NOTES:
1. 3D SURFACE FROM PAUL JENSEN PROFESSIONAL LAND SURVEYOR, DATED 2-1-2018.
2. BOTTOM OF BORING INDICATES BEDROCK CONTACT.

CASE 2 - ORTHOGRAPHIC VIEW
LYONS TANK SLIDE
SANTA CRUZ, CALIFORNIA
SAN LORENZO VALLEY WATER DISTRICT

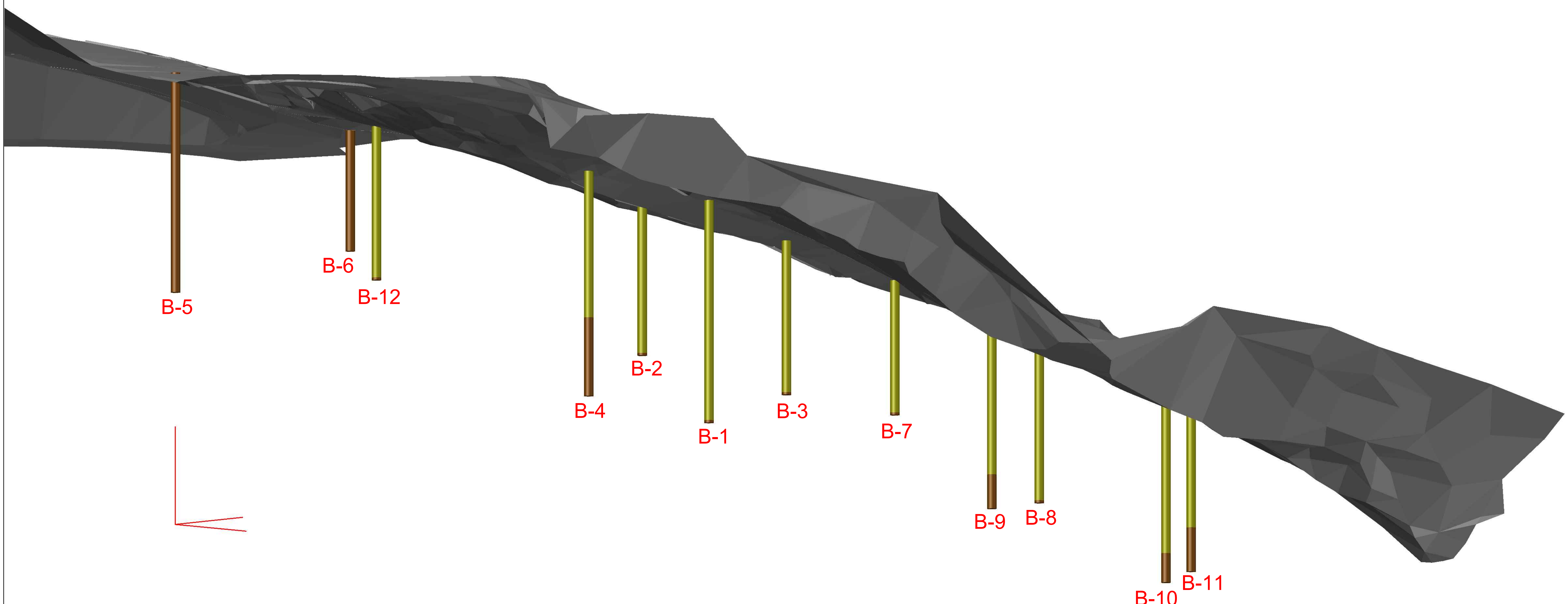
SCALE:	1" = 20'
DRAWN BY:	BRS
DATE:	MAY 2018
REVISED:	
JOB NO.	SC4090.1

HARO, KASUNICH & ASSOCIATES, INC.
GEOTECHNICAL AND COASTAL ENGINEERS
116 E. LAKE AVENUE, WATSONVILLE, CA 95076
(831) 722-4175

FIGURE NO. 57

SHEET NO.

0 5 10 20
SCALE: 1" = 20'



KEY:
- B-1 SOIL BORING LOCATION
- LANDSLIDE MATERIAL CASE 2
- NATIVE SOIL

NOTES:
1. 3D SURFACE FROM PAUL JENSEN PROFESSIONAL LAND SURVEYOR, DATED 2-1-2018.
2. BOTTOM OF BORING INDICATES BEDROCK CONTACT.

CASE 2 - ORTHOGRAPHIC VIEW
LYONS TANK SLIDE
SANTA CRUZ, CALIFORNIA
SAN LORENZO VALLEY WATER DISTRICT

SCALE:	1" = 20'
DRAWN BY:	BRS
DATE:	MAY 2018
REVISED:	
JOB NO.	SC4090.1

HARO, KASUNICH & ASSOCIATES, INC.
GEOTECHNICAL AND COASTAL ENGINEERS
116 E. LAKE AVENUE, WATSONVILLE, CA 95076
(831) 722-4175

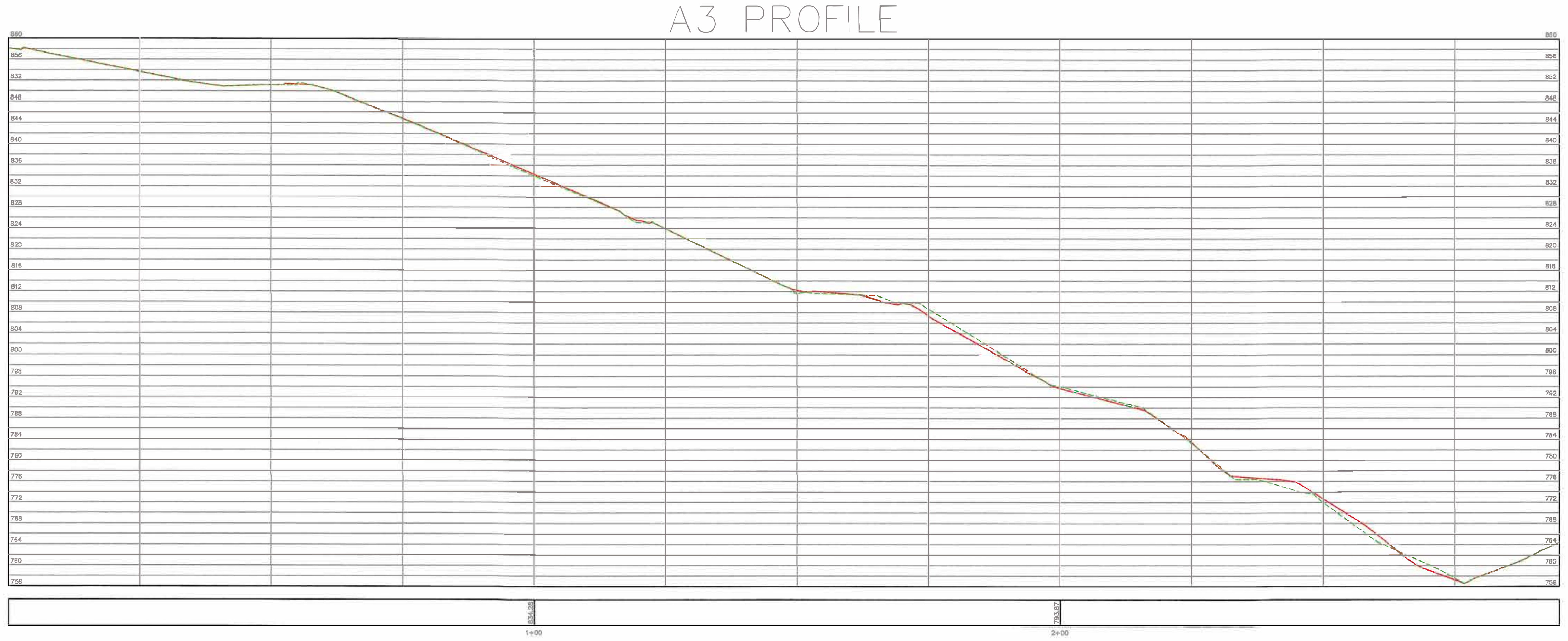
FIGURE NO. 58

SHEET NO.

APPENDIX C

Summary Results of Stability Analysis (Figures 59 – 71)

A3 PROFILE

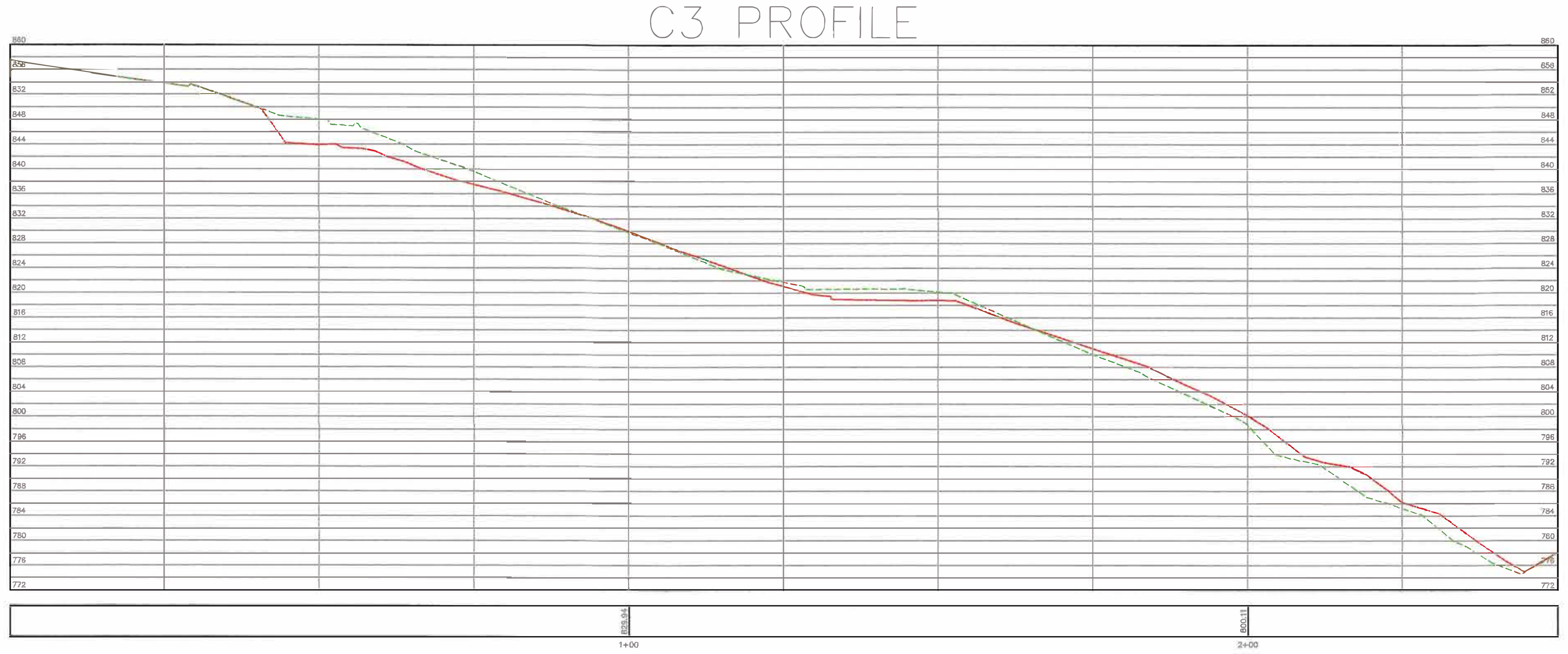


D3 PROFILE

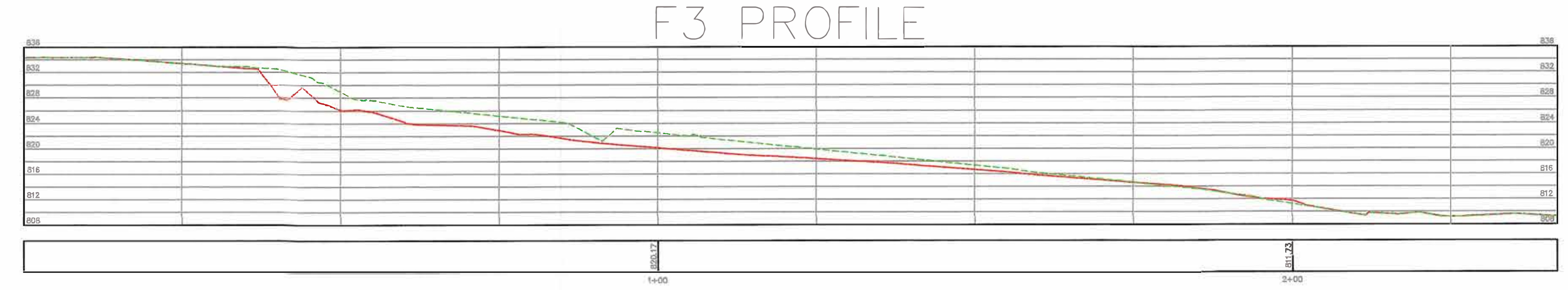
B3 PROFILE

E3 PROFILE

C3 PROFILE



F3 PROFILE



= CROSS SECTION PER FEBRUARY 25, 2017 SURVEY

= CROSS SECTION PER JUNE 03, 2017 SURVEY

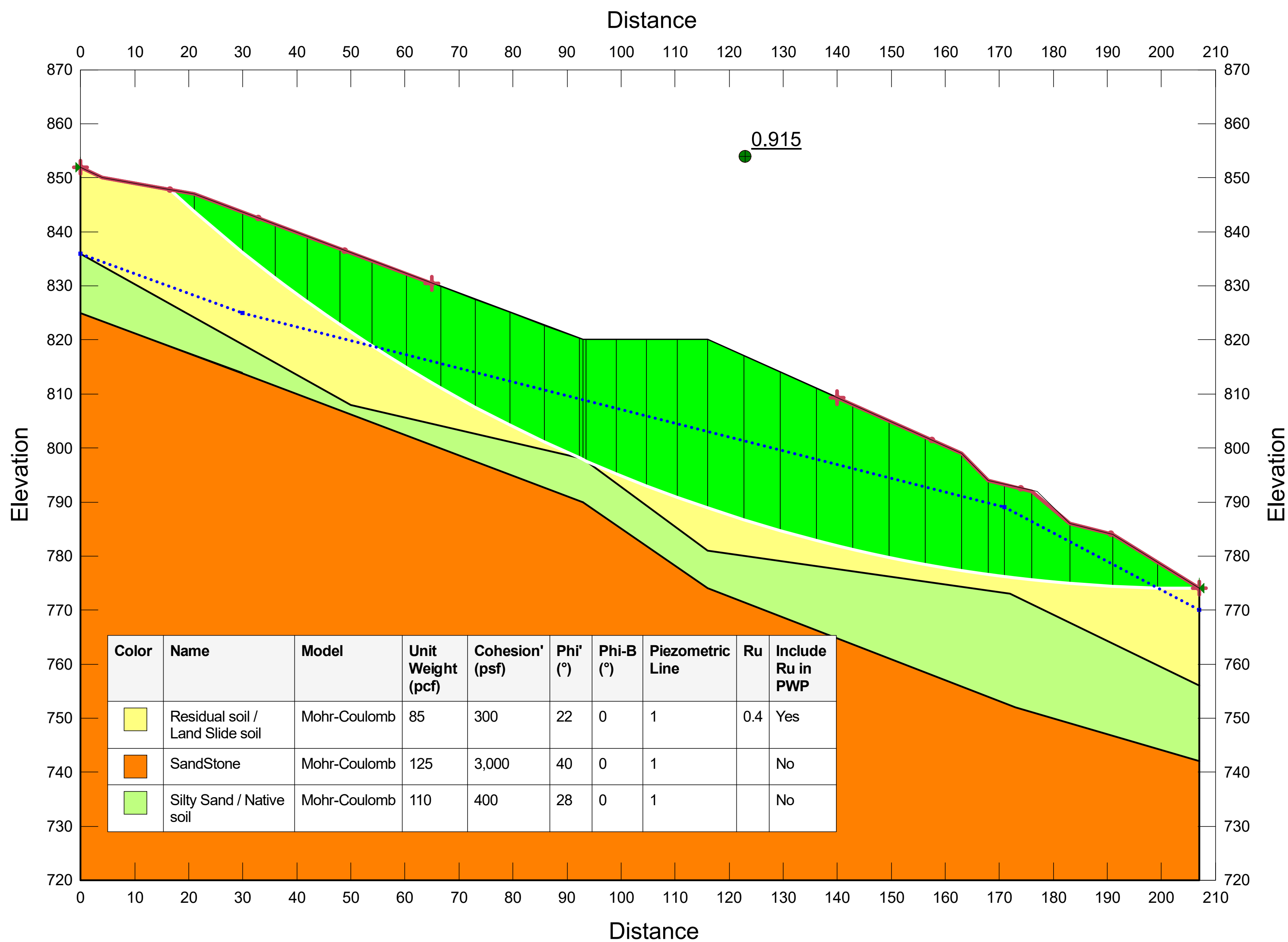
CROSS SECTIONS
OF THE LANDS OF
SAN LORENZO VALLEY
WATER DISTRICT
LYON WATER TREATMENT PLANT
365 MADRONE DRIVE
BOULDER CREEK, CALIFORNIA
APN 081-011-07
JUNE, 2017

H: 1"=20'
V: 1"=10'

PREPARED BY
PAUL JENSEN
PROFESSIONAL LAND SURVEYOR # 4627
SANTA CRUZ, CALIFORNIA

FIGURE NO. 59

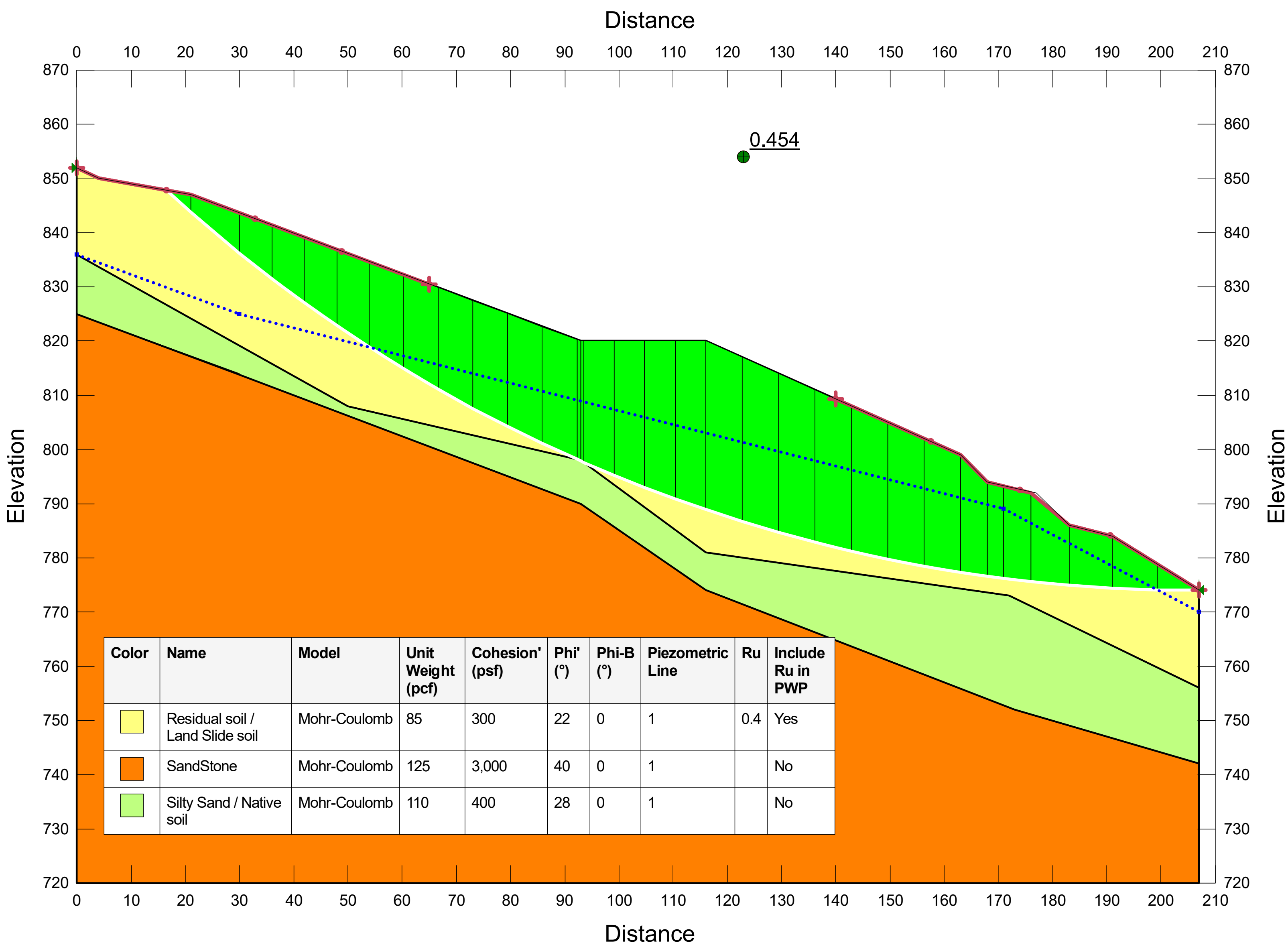
Lyon Tank Slope Stability Safety Factor- Cross Section C3- 2017 Landslide Condition - Static



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Piezometric Line	Ru	Include Ru in PWP
	Residual soil / Land Slide soil	Mohr-Coulomb	85	300	22	0	1	0.4	Yes
	SandStone	Mohr-Coulomb	125	3,000	40	0	1		No
	Silty Sand / Native soil	Mohr-Coulomb	110	400	28	0	1		No

Figure No. 60

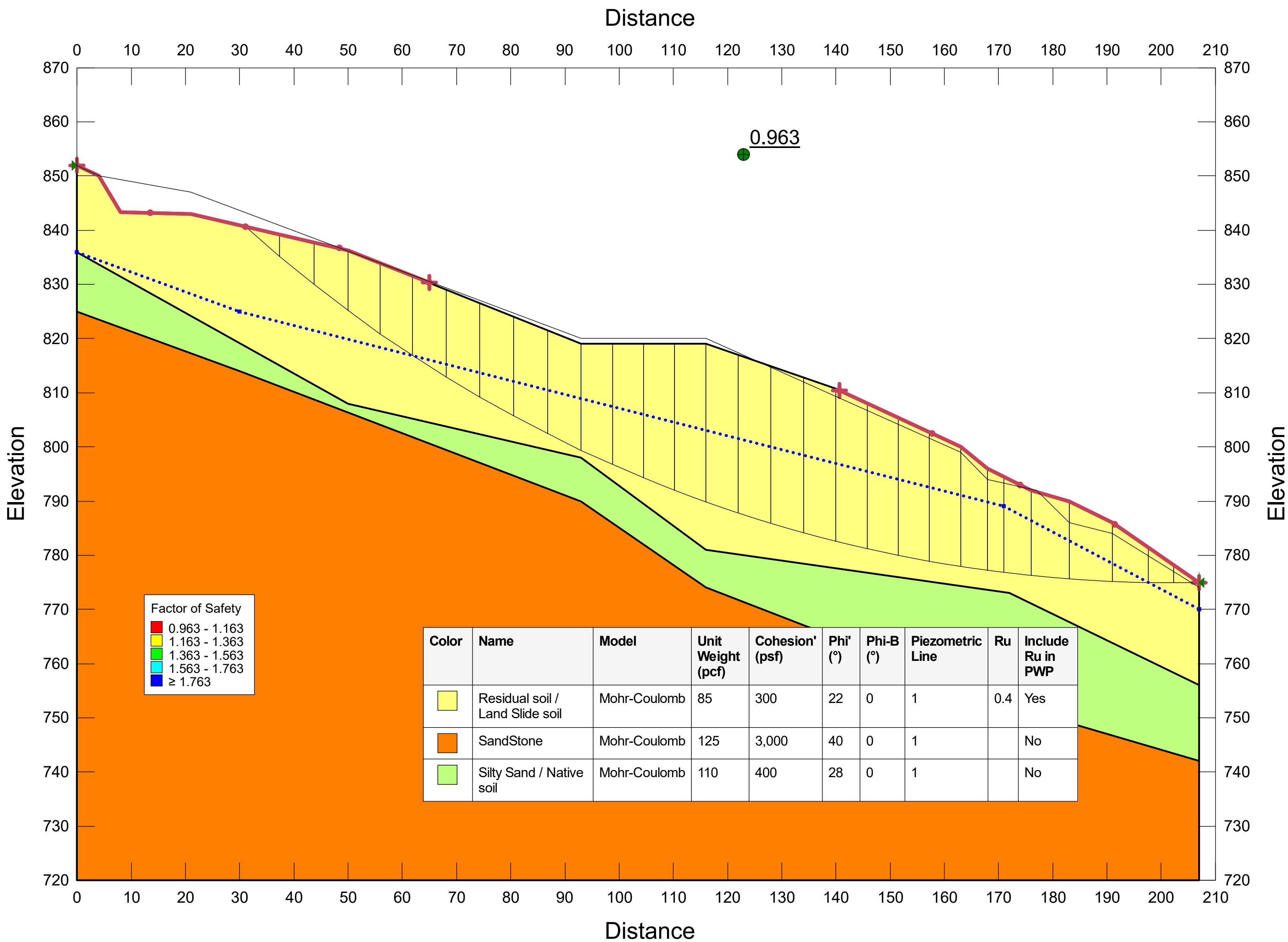
Lyon Tank Slope Stability Safety Factor- Cross Section C3- 2017 Landslide Condition - Seismic



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Piezometric Line	Ru	Include Ru in PWP
	Residual soil / Land Slide soil	Mohr-Coulomb	85	300	22	0	1	0.4	Yes
	SandStone	Mohr-Coulomb	125	3,000	40	0	1		No
	Silty Sand / Native soil	Mohr-Coulomb	110	400	28	0	1		No

Figure No. 61

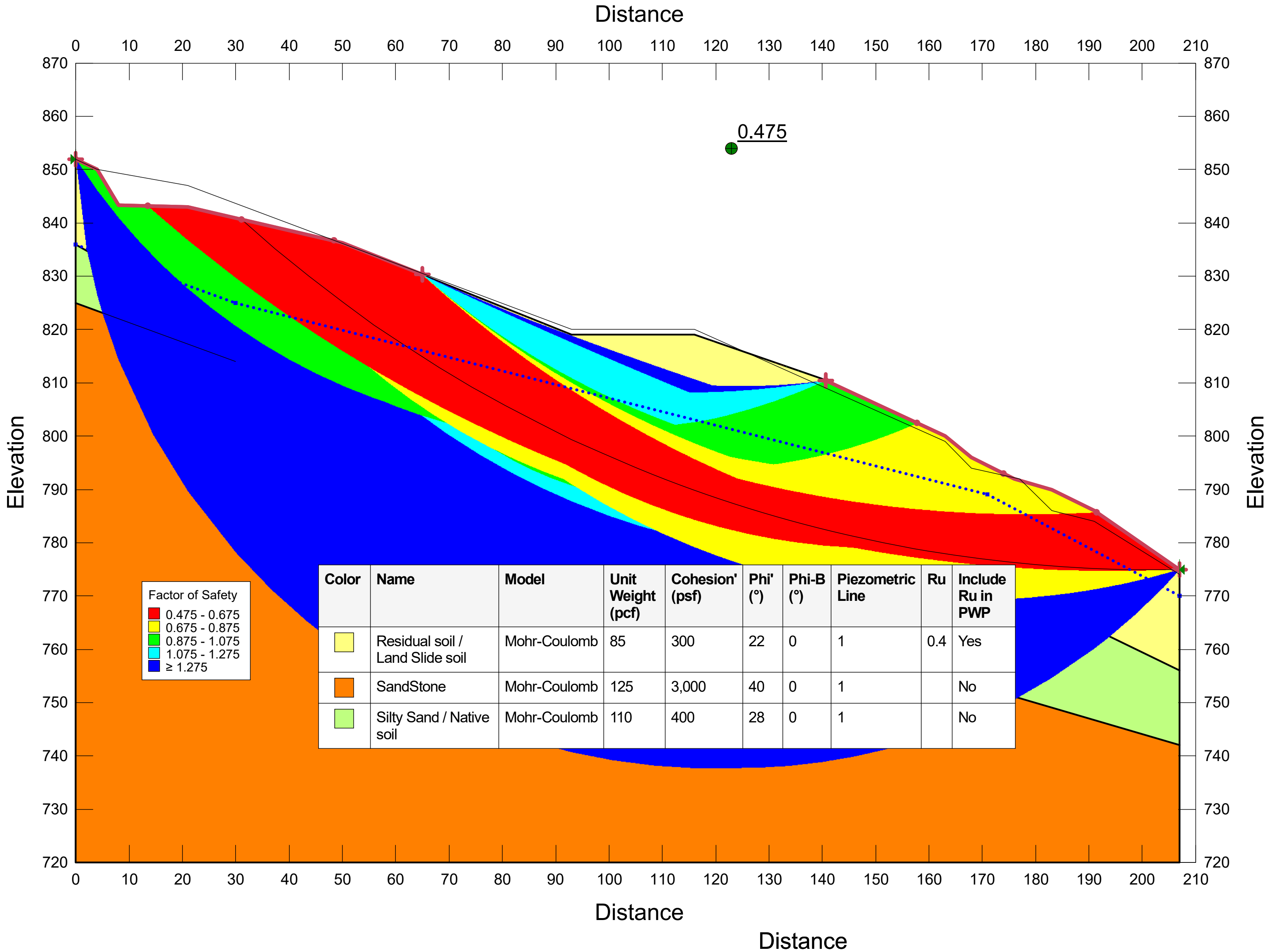
Lyon Tank Slope Stability Safety Factor- Cross Section C3- Current Condition - Static



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Piezometric Line	Ru	Include Ru in PWP
	Residual soil / Land Slide soil	Mohr-Coulomb	85	300	22	0	1	0.4	Yes
	SandStone	Mohr-Coulomb	125	3,000	40	0	1		No
	Silty Sand / Native soil	Mohr-Coulomb	110	400	28	0	1		No

Figure No. 62

Lyon Tank Slope Stability Safety Factor- Cross Section C3- Current Condition - Seismic

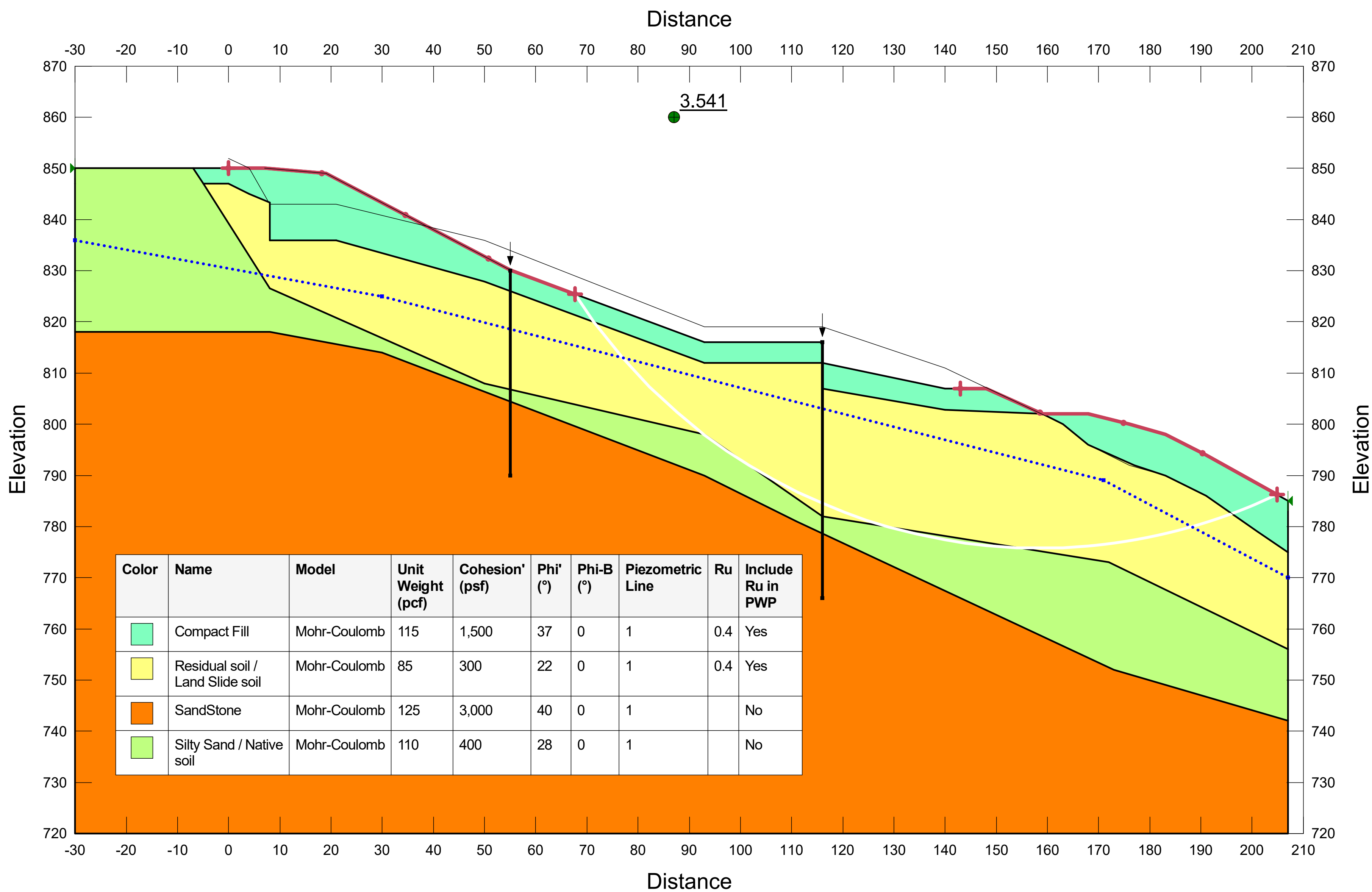


Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Piezometric Line	Ru	Include Ru in PWP
	Residual soil / Land Slide soil	Mohr-Coulomb	85	300	22	0	1	0.4	Yes
	SandStone	Mohr-Coulomb	125	3,000	40	0	1		No
	Silty Sand / Native soil	Mohr-Coulomb	110	400	28	0	1		No

Note : The border of light blue and dark blue zones shows the thickness of the future probable landslide. (SF less than 1.1 in seismic condition are considered as instable slope.

Figure No. 63

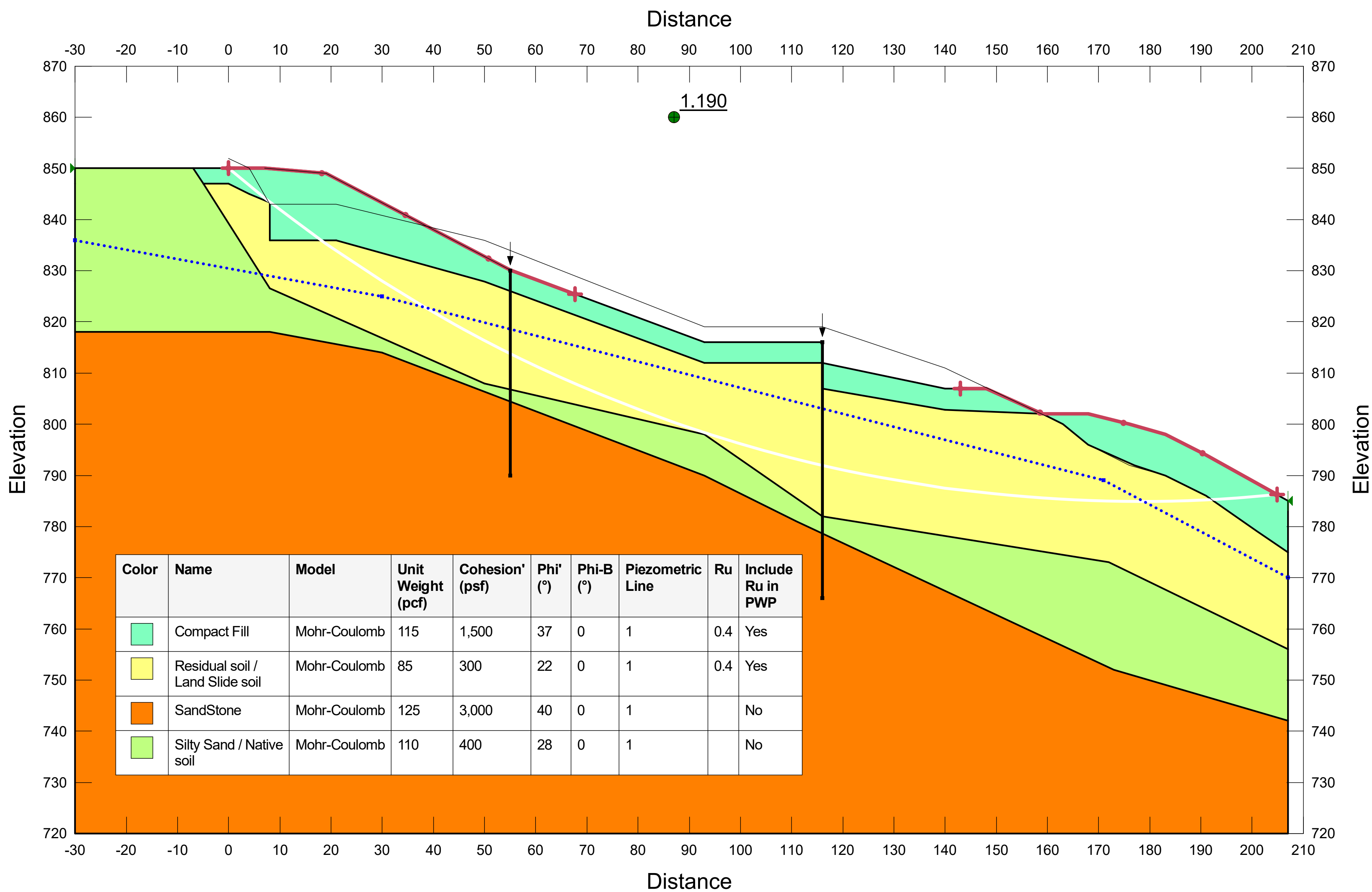
Lyon Tank Slope Stability Safety Factor- Cross Section C3- Improved Ground - Static



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Piezometric Line	Ru	Include Ru in PWP
	Compact Fill	Mohr-Coulomb	115	1,500	37	0	1	0.4	Yes
	Residual soil / Land Slide soil	Mohr-Coulomb	85	300	22	0	1	0.4	Yes
	SandStone	Mohr-Coulomb	125	3,000	40	0	1		No
	Silty Sand / Native soil	Mohr-Coulomb	110	400	28	0	1		No

Figure No. 64

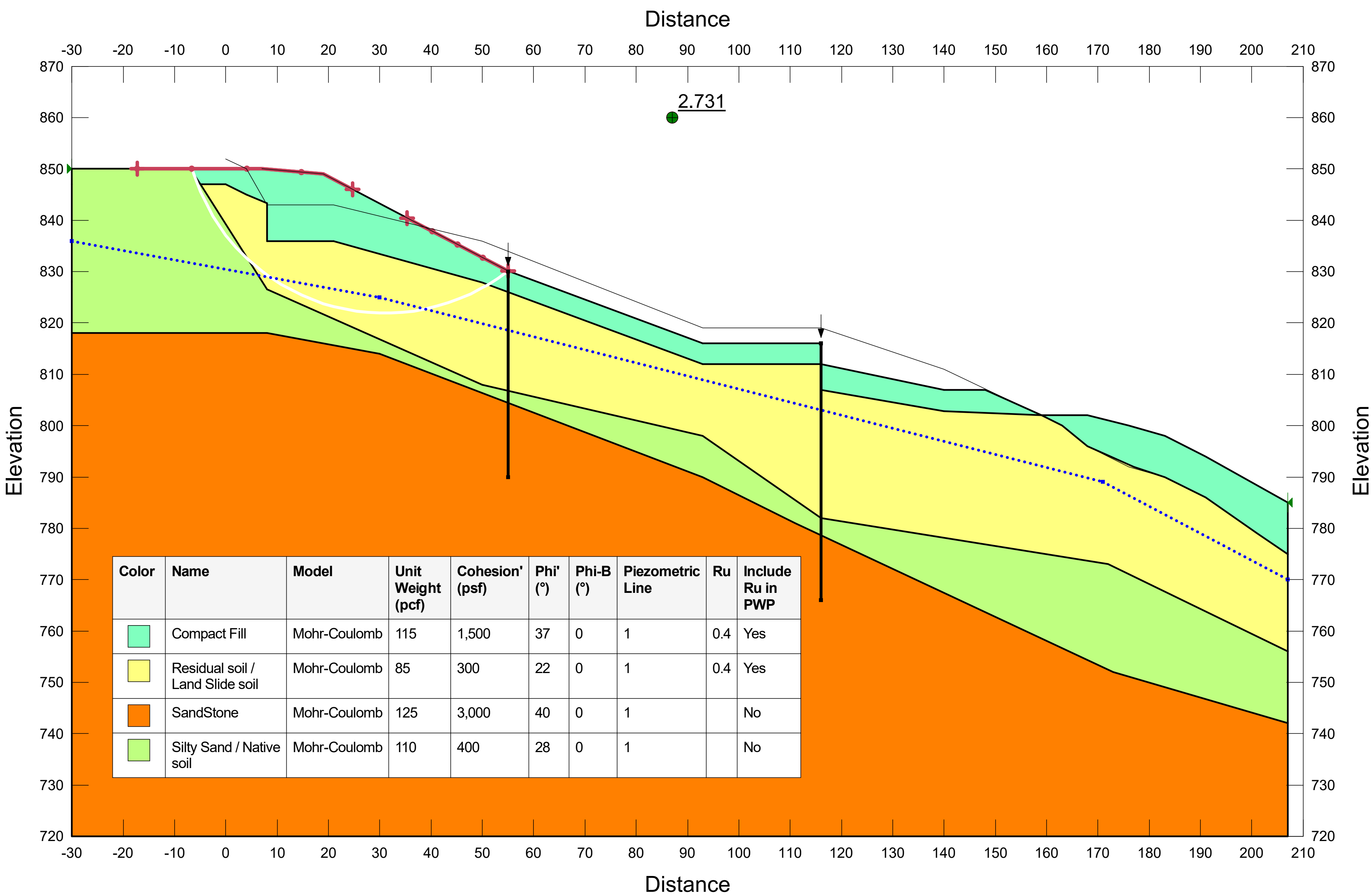
Lyon Tank Slope Stability Safety Factor- Cross Section C3- Improved Ground - Seismic



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Piezometric Line	Ru	Include Ru in PWP
	Compact Fill	Mohr-Coulomb	115	1,500	37	0	1	0.4	Yes
	Residual soil / Land Slide soil	Mohr-Coulomb	85	300	22	0	1	0.4	Yes
	SandStone	Mohr-Coulomb	125	3,000	40	0	1		No
	Silty Sand / Native soil	Mohr-Coulomb	110	400	28	0	1		No

Figure No. 65

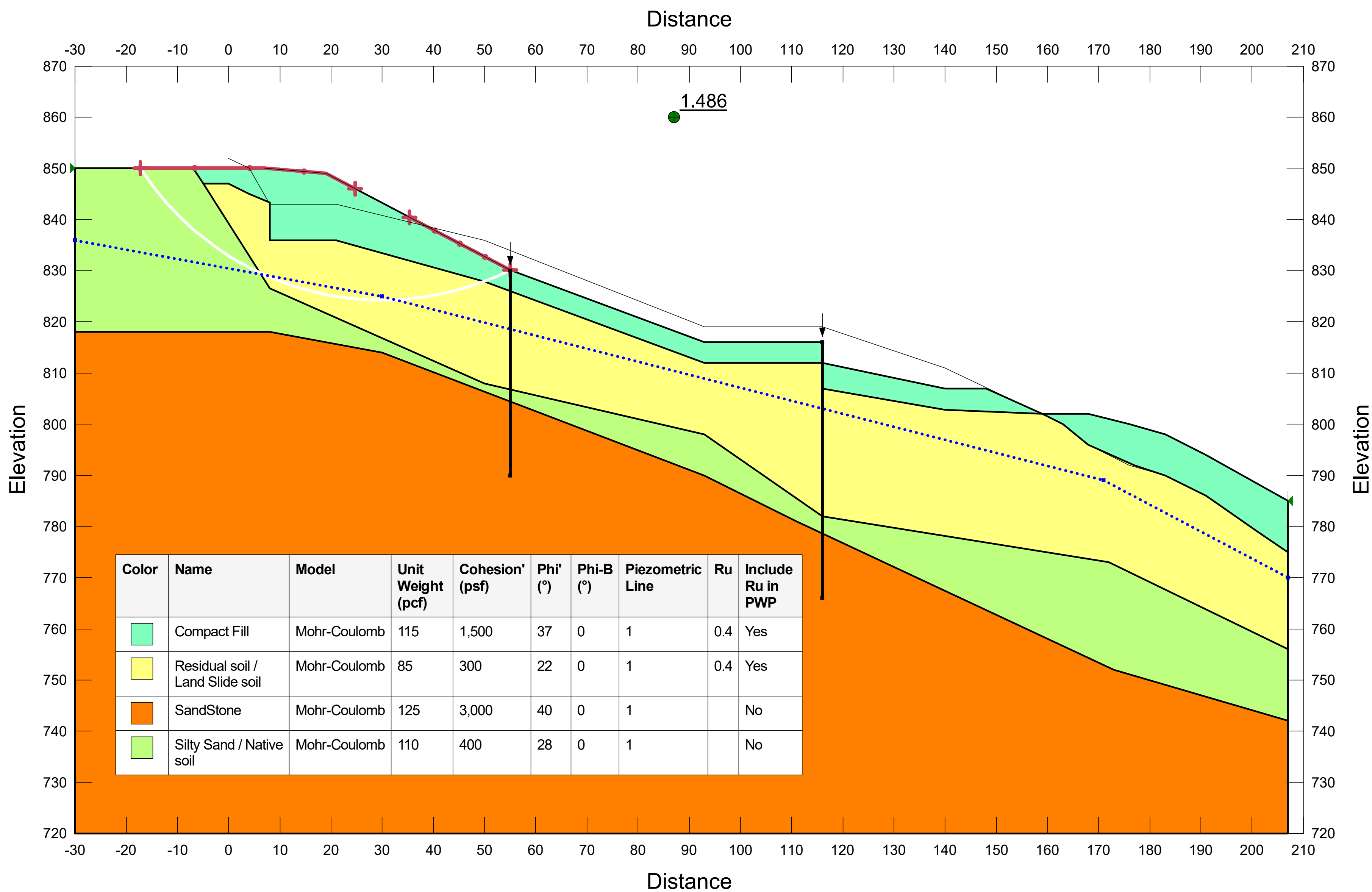
Lyon Tank Slope Stability Safety Factor- Cross Section C3- Improved Upper Road- Static



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Piezometric Line	Ru	Include Ru in PWP
	Compact Fill	Mohr-Coulomb	115	1,500	37	0	1	0.4	Yes
	Residual soil / Land Slide soil	Mohr-Coulomb	85	300	22	0	1	0.4	Yes
	SandStone	Mohr-Coulomb	125	3,000	40	0	1		No
	Silty Sand / Native soil	Mohr-Coulomb	110	400	28	0	1		No

Figure No. 66

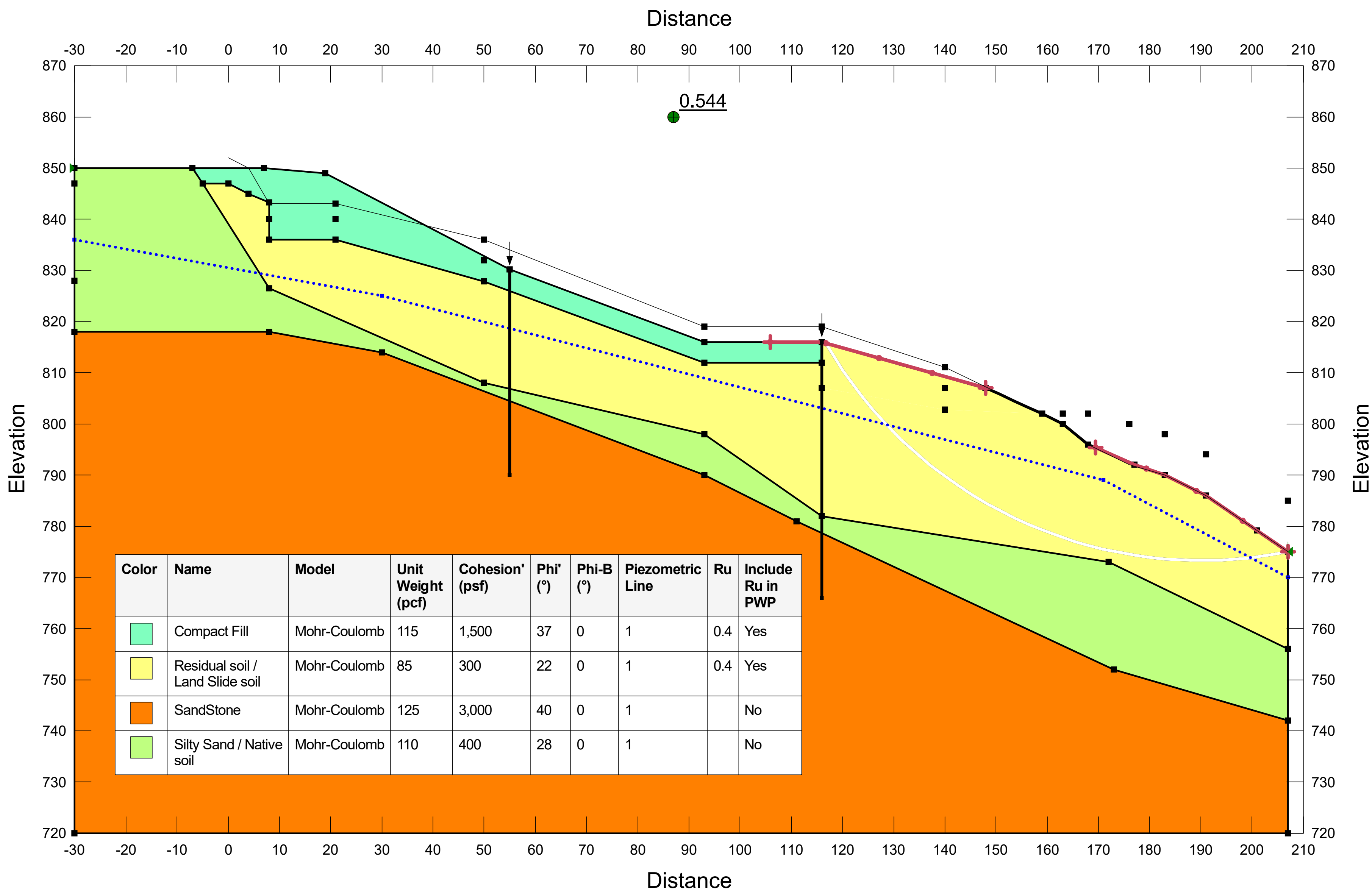
Lyon Tank Slope Stability Safety Factor- Cross Section C3- Improved Upper Road- Seismic



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Piezometric Line	Ru	Include Ru in PWP
	Compact Fill	Mohr-Coulomb	115	1,500	37	0	1	0.4	Yes
	Residual soil / Land Slide soil	Mohr-Coulomb	85	300	22	0	1	0.4	Yes
	SandStone	Mohr-Coulomb	125	3,000	40	0	1		No
	Silty Sand / Native soil	Mohr-Coulomb	110	400	28	0	1		No

Figure No. 67

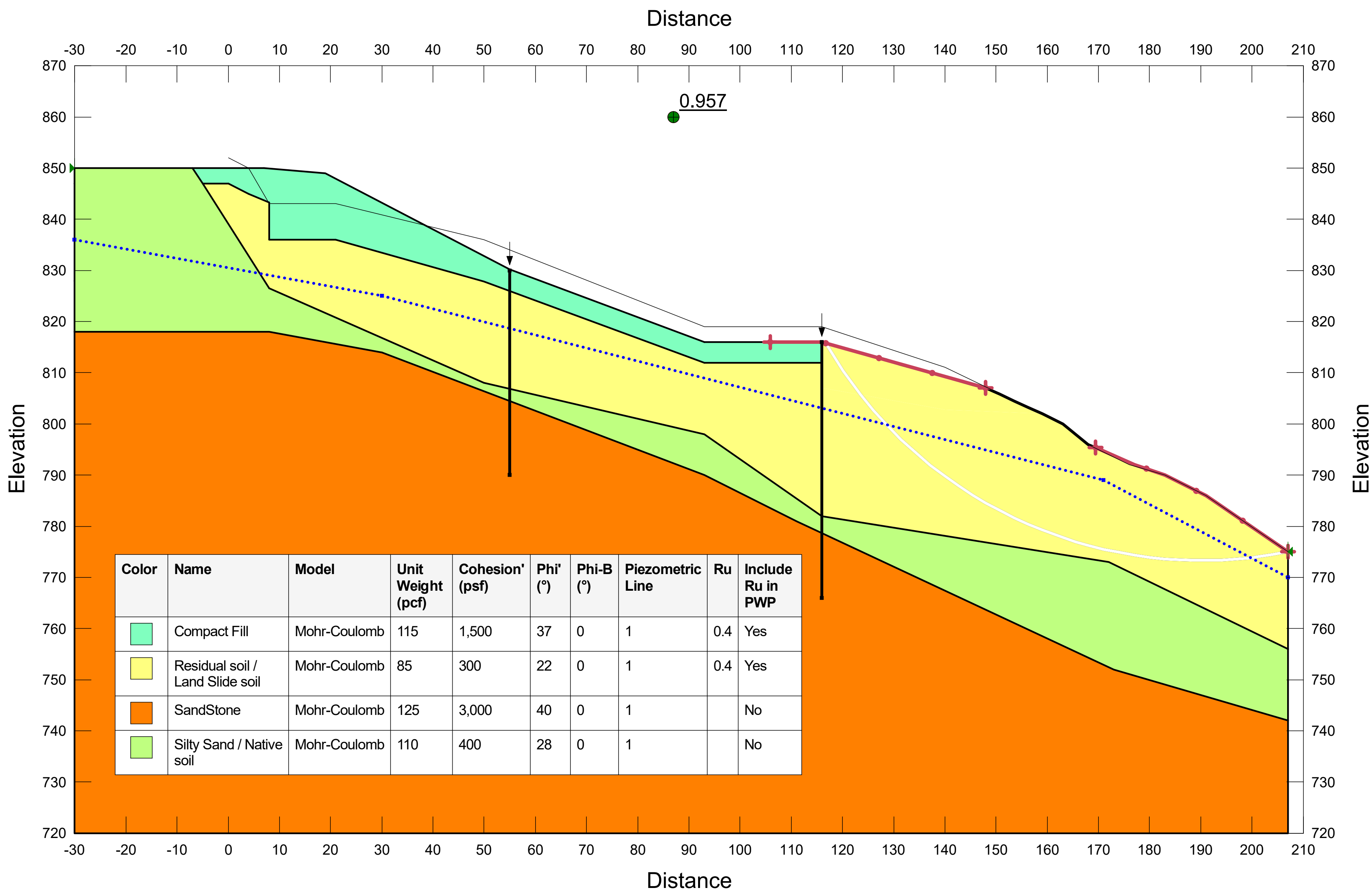
Lyon Tank Slope Stability Safety Factor - Cross Section C3 - Down Slope Lower Secant Row - Seismic



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Piezometric Line	Ru	Include Ru in PWP
	Compact Fill	Mohr-Coulomb	115	1,500	37	0	1	0.4	Yes
	Residual soil / Land Slide soil	Mohr-Coulomb	85	300	22	0	1	0.4	Yes
	SandStone	Mohr-Coulomb	125	3,000	40	0	1		No
	Silty Sand / Native soil	Mohr-Coulomb	110	400	28	0	1		No

Figure No. 68

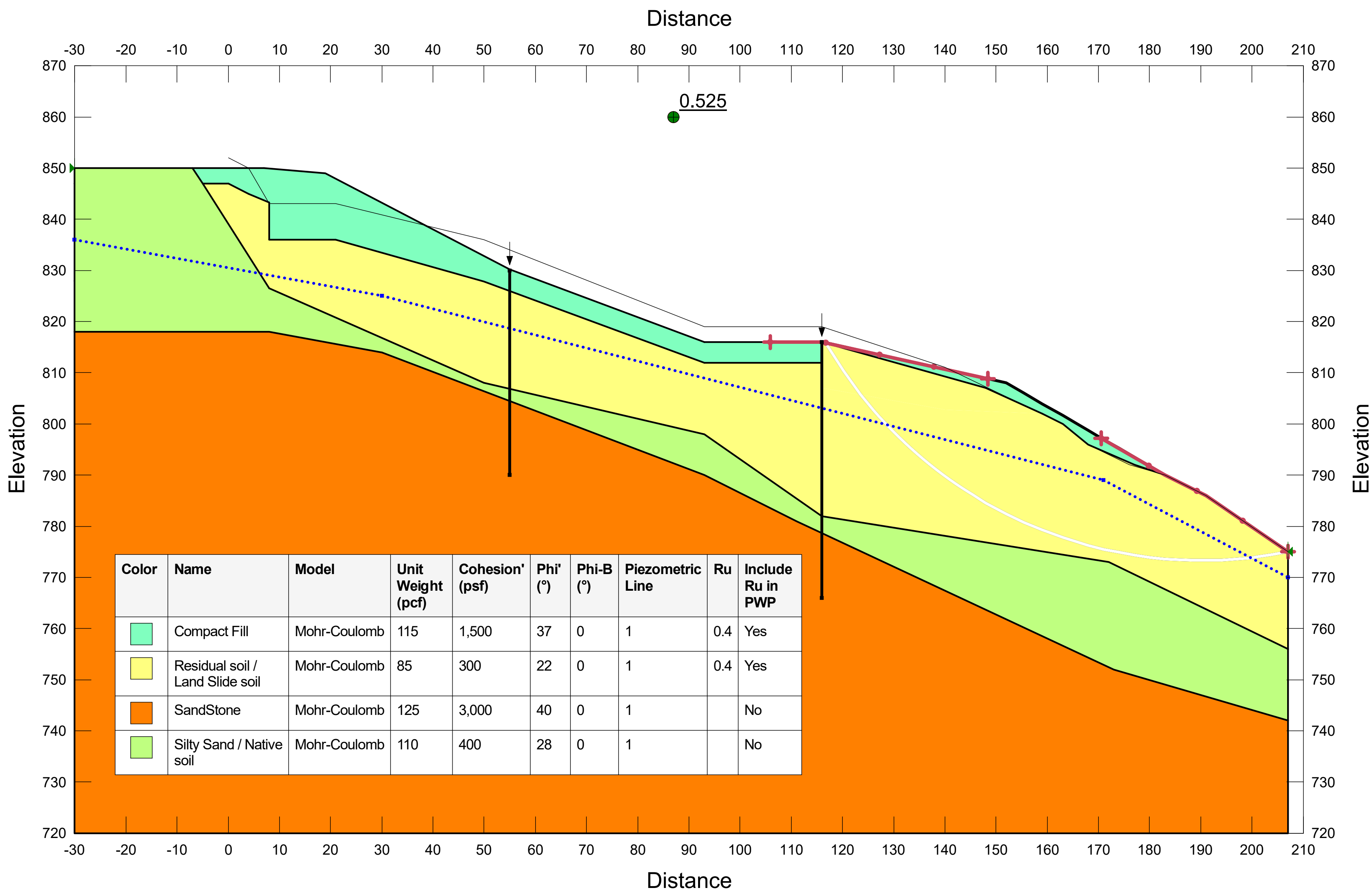
Lyon Tank Slope Stability Safety Factor - Cross Section C3 - Down Slope Lower Secant Row - Static



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Piezometric Line	Ru	Include Ru in PWP
	Compact Fill	Mohr-Coulomb	115	1,500	37	0	1	0.4	Yes
	Residual soil / Land Slide soil	Mohr-Coulomb	85	300	22	0	1	0.4	Yes
	SandStone	Mohr-Coulomb	125	3,000	40	0	1		No
	Silty Sand / Native soil	Mohr-Coulomb	110	400	28	0	1		No

Figure No. 69

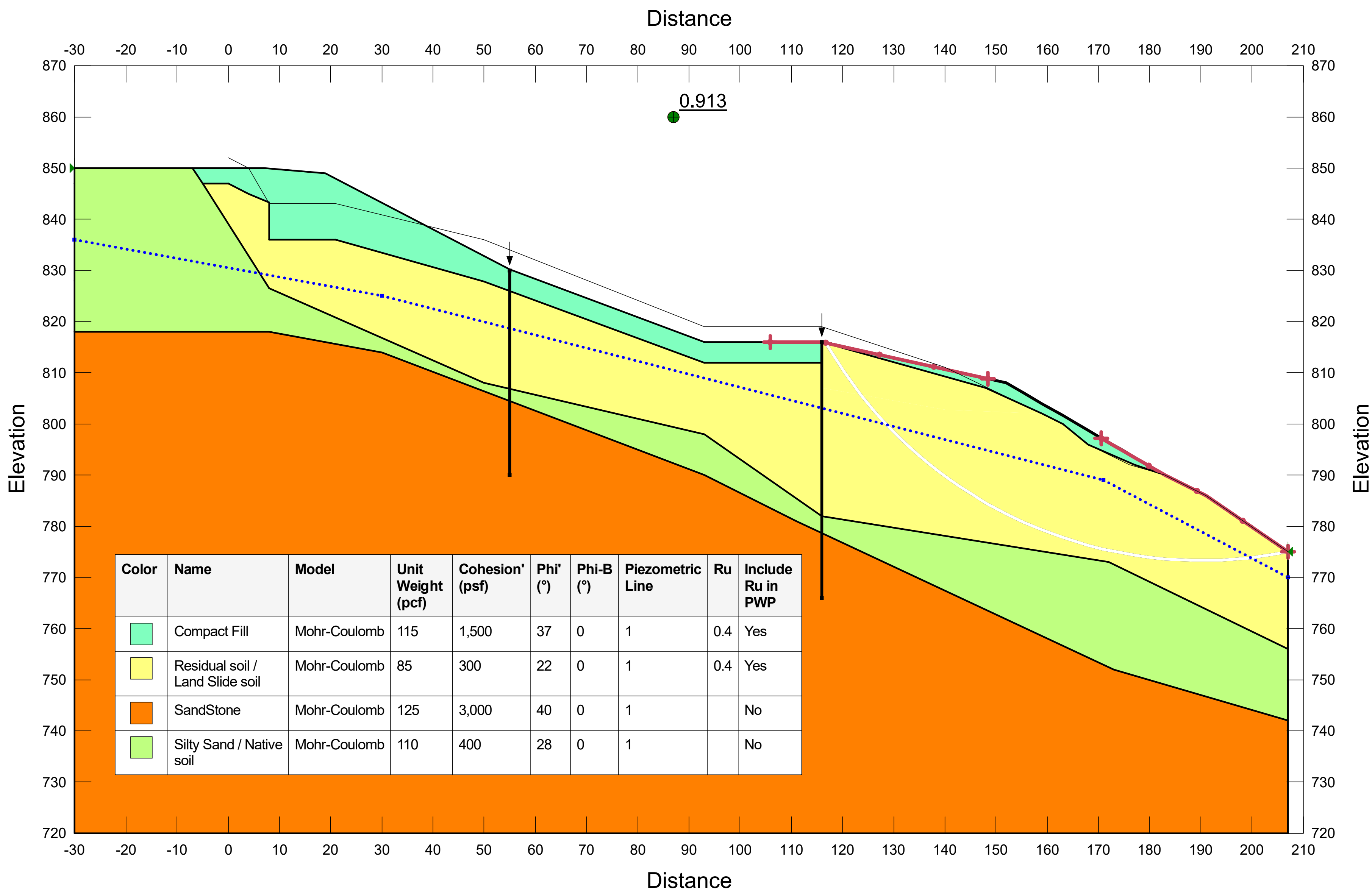
Lyon Tank Slope Stability Safety Factor - Cross Section C3 - Down Slope Lower Secant Row-With Grading - Seismic



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Piezometric Line	Ru	Include Ru in PWP
	Compact Fill	Mohr-Coulomb	115	1,500	37	0	1	0.4	Yes
	Residual soil / Land Slide soil	Mohr-Coulomb	85	300	22	0	1	0.4	Yes
	SandStone	Mohr-Coulomb	125	3,000	40	0	1		No
	Silty Sand / Native soil	Mohr-Coulomb	110	400	28	0	1		No

Figure No. 70

Lyon Tank Slope Stability Safety Factor - Cross Section C3 - Down Slope Lower Secant Row- With Grading - Static



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Piezometric Line	Ru	Include Ru in PWP
	Compact Fill	Mohr-Coulomb	115	1,500	37	0	1	0.4	Yes
	Residual soil / Land Slide soil	Mohr-Coulomb	85	300	22	0	1	0.4	Yes
	SandStone	Mohr-Coulomb	125	3,000	40	0	1		No
	Silty Sand / Native soil	Mohr-Coulomb	110	400	28	0	1		No

Figure No. 71

Project No. SC4090.1
6 August 2018

APPENDIX D

Some Photos From The Project Site