Appendix D
Geotechnical Engineering Investigation

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Celebrating 50 YEARS OF SERVICE 1971-2021

February 16, 2022
File Number 22119

Chandler Partners
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Suite 203
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Attention: William Hiestand

Subject: Geotechnical Engineering Investigation
Proposed Residential Development
23755 Newhall Avenue, Santa Clarita, California

Dear Mr. Hiestand:

This letter transmits the Geotechnical Engineering Investigation for the subject site prepared by Geotechnologies, Inc. This report provides geotechnical recommendations for the development of the site, including earthwork, seismic design, retaining walls, excavations and foundation design. Engineering for the proposed project should not begin until approval of the geotechnical investigation is granted by the local building official. Significant changes in the geotechnical recommendations may result due to the building department review process.

The validity of the recommendations presented herein is dependent upon review of the geotechnical aspects of the project during construction by this firm. The subsurface conditions described herein have been projected from limited subsurface exploration and laboratory testing. The exploration and testing presented in this report should in no way be construed to reflect any variations which may occur between the exploration locations, or which may result from changes in subsurface conditions.

Should you have any questions please contact this office.

Respectfully submitted,
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GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
23755 NEWHALL AVENUE
SANTA CLARITA, CALIFORNIA

INTRODUCTION

This report presents the results of the geotechnical engineering investigation performed on the subject site. The purpose of this investigation was to identify the distribution and engineering properties of the geologic materials underlying the site, and to provide geotechnical recommendations for the design of the proposed development.

This investigation included the excavation of fourteen borings and ten test pits, collection of representative samples, laboratory testing, engineering analysis, review of published geologic data, review of available geotechnical engineering information and the preparation of this report. The exploratory excavation locations are shown on the enclosed Geologic Map and Proposed Site Plan. The results of the exploration and the laboratory testing are presented in the Appendix of this report.

PROPOSED DEVELOPMENT

Information concerning the proposed development was furnished by the client and the office of Alliance Land Planning and Engineering, Inc. In addition, the Site Development Plan prepared by Alliance Land Planning and Engineering, Inc., dated August 20, 2021, was reviewed for the preparation of this report. The proposed residential development will include the construction of 2 multi-family residential structures, and approximately 35 townhome units which will be clustered in 7 structures. The proposed structures are expected to range between two and four stories in height, and will be built at-grade. In addition, a single-story recreation center, a pool, and several car port structures are being proposed. The location and alignment of the proposed structures is shown in the enclosed Proposed Site Plan.

Grading of the site will consist of the cutting of some of the existing slopes, and the filling of some of the low areas, for the creation of level terraces. The proposed cut-slopes will be up to 36 feet in vertical height, and will be cut at a maximum slope gradient of 2:1 (horizontal:vertical). It is anticipated that up to 15 feet of fill soils will be placed within the lower areas.

In addition, several retaining walls are proposed throughout the site, to aid in the creation of the proposed level terraces. These retaining walls are anticipated to range from 4 to 18 feet in height. The location and height of the retaining walls are shown in the enclosed Proposed Site Plan. The proposed ground elevations are illustrated in the enclosed Cross Sections A-A', B-B', C-C', D-D' and E-E'.

Any changes in the design of the project or location of any structure, as outlined in this report, should be reviewed by this office. The recommendations contained in this report should not be considered valid until reviewed and modified or reaffirmed, in writing, subsequent to such review.

SITE CONDITIONS

The project site is located at 23755 Newhall Avenue, in the City of Santa Clarita, California. The site is approximately 10 acres in area, bounded by Newhall Avenue to the northeast, a commercial development and undeveloped hillside land to the southeast, an industrial development currently under construction to the southwest and northwest, and a medical care development to the northwest. The site is shown relative to nearby topographic features in the enclosed Vicinity Map.

As illustrated in the enclosed Geologic Map, the existing site topography consists of a canyon found near the middle of the site, and two ridges which ascend to the south and west on each side of the canyon. The canyon is oriented generally along the north-south direction, and descends toward Newhall Avenue to the north. The portion of the site located along Newhall Avenue is relatively level.

Within the site limits, the highest elevation observed at the eastern ridge is 1,440 feet. The highest elevation observed at the western ridge is 1,380 feet. The elevation at the canyon found within the central portion of the site ranges from 1,317 feet at the northern corner to 1,353 at the south. The existing slope gradients range from approximately 1½:1 to 4:1 (horizontal:vertical). The steepest slope gradient was observed within the western corner of the site, for a slope which will be redefined as part of the proposed project at a 2:1 slope gradient.

The majority of the site is undeveloped, with the exception of the relatively level portion of the site located along Newhall Avenue. The observed development consists of a single-story commercial building, two single-story pre-fabricated structures, and paved parking areas. Vegetation at the site consists of numerous mature trees, as well as bushes, shrubs, and seasonal grasses. Drainage at the site appears to be by sheetflow to Newhall Avenue to the north.

LOCAL GEOLOGY

The site is located in an area of low hills between the San Gabriel and the Santa Susana Mountains. This area is characterized by relatively young, deformed fluvial-origin sedimentary rocks of the Saugus Formation. The Saugus Formation consists of sandstone, conglomerate and thin beds of siltstone and claystone. The canyon bottoms are filled with alluvial sediments derived from erosion of the adjacent hills. The geology of the site vicinity is presented on the enclosed Local Geologic Maps.

RESEARCH

R.T. Frankian & Associates, January 30, 2020, Geotechnical Plan Review – Grading Plan, Needham Ranch, Phase 2, Tract Nos. 50283-03 and -04, Santa Clarita, California, Job Number 99-506-021.

This firm is in receipt of the referenced geotechnical report, which was prepared for the neighboring site located to the southwest and northwest of the project site. This report pertains to the geotechnical review of the grading plans prepared for the industrial development currently under construction. A total of 38 exploratory excavations were performed for the preparation of this report. One of these exploratory excavations (Boring B42) was excavated in the vicinity of the project site, near the western corner. The location of this boring is shown in the enclosed Geologic Map, and a copy of its log has been included in the Appendix.

Based on review of this report, and recent site observations performed by representatives of our firm, it is our understanding that the ridge which ascends to the west of the project site was recently trimmed for the creation of an upper level terrace. Within the adjacent site, the top of this ridge previously reached an elevation of approximately 1,486 feet. After the ridge was trimmed, the elevation of the newly created terrace appears to be in the order of elevation 1,380 feet. The grading of this ridge extended into a small portion of the southwestern corner of the project site. The enclosed Geologic Map and Cross Sections B-B' and C-C' illustrate the approximate topographic elevations after the trimming of this ridge.

GEOTECHNICAL EXPLORATION

FIELD EXPLORATION

The site was explored on April 19, 26, 27, 29 and 30, September 30 and October 1, 2021. Fourteen borings and ten test pits were excavated as part of the exploration. The table below summarizes the exploratory excavations conducted at the site:

Type of Excavation	Method of Excavation	Number of Excavations	Excavation Number	Depth Range
Borings	8-inch Hollowstem Auger	5	B1, B2, B3, B4 and B5	20 to 50
	24-inch Bucket Auger	4	B6, B7, B8 and B9	30 to 50
	24-inch Fly Auger	5	B10, B11, B12, B13 and B14	30 to 40
Test Pits	Hand Tools	4	TP1, TP2, TP3 and TP4	10
	Backhoe	6	TP5, TP6, TP7, TP8, TP9 and TP10	10 to 14

With the exception of the borings drilled with the hollowstem auger (B1 through B5), all other excavations were down-hole logged by a geologist. The exploration locations are shown on the Geologic Map and Proposed Site Plan, and the geologic materials encountered are logged on Plates A-1 through A-24.

The location of exploratory excavations was determined from hardscaped features shown in the enclosed Geologic Map. Elevations of the exploratory excavations were determined from elevations presented in a topographic survey provided by Alliance Land Planning and Engineering, Inc., not dated. The location and elevation of the exploratory excavations should be considered accurate only to the degree implied by the method used.

Geologic Materials

Geologic materials encountered in the boring and test pits include artificial fill, alluvial soils, older alluvial soils, and bedrock of the Saugus Formation.

Existing Artificial Fill (af)

Fill materials were encountered during exploration to depths ranging between 1 and 5 feet below the existing grade. The deeper fill was observed within the canyon and lower level areas. The fill observed within the hillsides generally consists of a thin mantle of slope soils which are 1 to 2 feet in thickness.

Fill materials encountered on the subject site consist of silty sands and sands, which were predominantly dark brown in color, slightly moist to moist, medium dense, and fine to medium grained, with occasional cobbles.

Alluvium (Qa)

Alluvium was observed underlying the fill in the excavations conducted within the canyon and lower level areas. The alluvium consists of silty sands and poorly graded sands, which are generally dark brown in color, moist, medium dense to very dense, and fine to coarse grained, with occasional cobbles.

Older Alluvium (Qoa)

Older alluvium was encountered in all the exploratory excavations, underlying the fill or the alluvial soil. These natural soils consist generally of silty sands, and poorly graded and well graded sands. The older alluvial soils are yellowish to dark brown, moist, dense to very dense, and fine to coarse grained, with varying amounts of cobbles and occasional pebbles.

Saugus Formation Bedrock (QTs)

Sedimentary bedrock of the Saugus Formation was observed underlying the older alluvium in Borings B6, B7, B8, B9, B10, B11, B12, B13 and B14. In these borings, the bedrock was observed at depths ranging between 15 and 35 feet below the existing grade. The bedrock was not observed in the remaining excavations. Furthermore, the bedrock was not observed to be exposed on any of the existing ridges.

The observed bedrock consist primarily of sandstone and conglomerate, with occasional siltstone layers. The bedrock ranged from yellowish brown to light gray to grayish brown in color, and it is slightly moist, moderately hard to hard, and fine to coarse grained, with occasional pebbles and cobbles.

The bedrock was observed to be moderately to poorly bedded. The bedding identified in the borings dips to the northwest, at angles ranging between 10 and 20 degrees. These bedding orientations are consistent with the bedding mapped and presented in the enclosed Local Geologic Maps.

Groundwater

Groundwater was not encountered during exploration, conducted to a maximum depth of 50 feet below the lowest site grade. According to the Seismic Hazard Zone Report for the Oat Mountain 7½-Minute Quadrangle (CDMG SHZR 05, 1997, revised 2006), the historically highest groundwater level for the site is expected to range between 40 and 60 feet below the existing grade observed along Newhall Avenue. A copy of the historic groundwater contour map is provided in the Appendix of this report.

Fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, and other factors not evident at the time of the measurements reported herein. Fluctuations also may occur across the site. High groundwater levels can result in changed conditions.

Caving

Caving was not experienced during exploration.

OIL FIELDS AND OIL WELLS

Based on review of the Interactive Well Finder Map Application, developed by the California Department of Conservation, the site is located within the limits of the Newhall Oil Field. Furthermore, the map indicates that an oil well had been drilled within the central portion of the site. A copy of this map has been enclosed in the Appendix as the Oil Field and Oil Well Map.

The oil well drilled at the site is labeled "Jack L. Watkins Legion 1 Well". According to documents obtained from the California Department of Conservation Website, the well was abandoned in 1953. Copies of these abandonment documents may be found in the Appendix of this report. This firm recommends that an experienced consultant/contractor should be contacted to accurately locate well, and determine if it was properly abandoned.

SEISMIC EVALUATION

REGIONAL GEOLOGIC SETTING

The subject property is located in the Transverse Ranges Geomorphic Province. The Transverse Ranges are characterized by roughly east-west trending mountains and the northern and southern boundaries are formed by reverse fault scarps. The convergent deformational features of the Transverse Ranges are a result of north-south shortening due to plate tectonics. This has resulted in local folding and uplift of the mountains along with the propagation of thrust faults (including blind thrusts). The intervening valleys have been filled with sediments derived from the bordering mountains.

REGIONAL FAULTING

Based on criteria established by the California Division of Mines and Geology (CDMG) now called California Geologic Survey (CGS), Faults may be categorized as Holocene-active, Pre-Holocene faults, and Age-undetermined faults. Holocene-active faults are those which show evidence of surface displacement within the last 11,700 years. Pre-Holocene faults are those that have not moved in the past 11,700 years. Age-undetermined faults are faults where the recency of fault movement has not been determined.

Buried thrust faults are faults without a surface expression but are a significant source of seismic activity. They are typically broadly defined based on the analysis of seismic wave recordings of hundreds of small and large earthquakes in the southern California area. Due to the buried nature of these thrust faults, their existence is usually not known until they produce an earthquake. The risk for surface rupture potential of these buried thrust faults is inferred to be low (Leighton, 1990). However, the seismic risk of these buried structures in terms of recurrence and maximum potential magnitude is not well established. Therefore, the potential for surface rupture on these surface-verging splays at magnitudes higher than 6.0 cannot be precluded.

Legion Fault

As illustrated in the enclosed Local Geologic Map – Winterer & Durham, the Legion Fault is mapped along the northern portion of the site, situated parallel to Newhall Avenue. This firm has reviewed the Geological Survey Professional Paper 334-H, titled "Geology of Southern Ventura Basin, Los Angeles County, California", prepared by E.L. Winterer and D.L. Durham (U.S. Department of the Interior, 1962). According to this publication, "The Legion Fault is named for its exposure behind the American Legion Hall, about 1 mile west of Newhall. The fault is mostly concealed by alluvium, but it is inferred to connect with a system of reverse faults exposed on the ridge south of Elsmere Canyon."

According to the California Geological Survey, the Legion Fault is considered a Pre-Quaternary Fault. No Special Studies Zones have been delineated by the State of California, or the County of Los Angeles, along any part of the Legion Fault. Based on its concealment under the alluvium, and the age of its most recent displacement (pre-quaternary), it is the opinion of this firm that the potential for surface rupture at the site due to this fault is considered remote.

SEISMIC HAZARDS AND DESIGN CONSIDERATIONS

The primary geologic hazard at the site is moderate to strong ground motion (acceleration) caused by an earthquake on any of the local or regional faults. The potential for other earthquake-induced hazards was also evaluated including surface rupture, liquefaction, dynamic settlement, inundation and landsliding.

Surface Rupture

In 1972, the Alquist-Priolo Special Studies Zones Act (now known as the Alquist-Priolo Earthquake Fault Zoning Act) was passed into law. As revised in 2018, The Act defines "Holocene-active" Faults utilizing the same aging criteria as that used by California Geological Survey (CGS). However, established state policy has been to zone only those faults which have direct evidence of movement within the last 11,700 years. It is this recency of fault movement that the CGS considers as a characteristic for faults that have a relatively high potential for ground rupture in the future.

CGS policy is to delineate a boundary from 200 to 500 feet wide on each side of the Holocene-Active fault trace based on the location precision, the complexity, or the regional significance of the fault. If a site lies within an Earthquake Fault Zone, a geologic fault rupture investigation must be performed that demonstrates that the proposed building site is not threatened by surface displacement from the fault before development permits may be issued.

Ground rupture is defined as surface displacement which occurs along the surface trace of the causative fault during an earthquake. Based on research of available literature and results of site reconnaissance, no known Holocene-active or Pre-Holocene faults underlie the subject site. In addition, the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. Review of the CGS Map, Earthquake Zones of Required Investigation of the Oat Mountain Quadrangle, indicates that the subject site is not located within an Alquist-Priolo Earthquake Fault Zone (CGS, 1999 and 2002). The nearest Fault Zone is located approximately 2 miles to the south of the site, and corresponds to the Sierra Madre Fault.

Based on these considerations, the potential for surface ground rupture at the subject site is considered low.

Liquefaction

Liquefaction is a phenomenon in which saturated silty to cohesionless soils below the groundwater table are subject to a temporary loss of strength due to the buildup of excess pore pressure during cyclic loading conditions such as those induced by an earthquake. Liquefaction-related effects include loss of bearing strength, amplified ground oscillations, lateral spreading, and flow failures.

Liquefaction typically occurs in areas where groundwater is less than 50 feet from the surface, and where the soils are composed of poorly consolidated, fine to medium-grained sand. In addition to the necessary soil conditions, the ground acceleration and duration of the earthquake must also be of a sufficient level to initiate liquefaction.

The Earthquake Zones of Required Investigation Maps of the State of California (CDMG, 1998), do not classify the site as part of a "Liquefiable" area. This determination is based on groundwater depth records, soil type and distance to a fault capable of producing a substantial earthquake. A copy of this map is provided in the Appendix.

Groundwater was not encountered during exploration, conducted to a maximum depth of 50 feet below the lowest site ground surface. Based on the Seismic Hazard Zone Report of the San Oat Mountain Quadrangle (CDMG SHZR 05, 1997, revised 2006), the historically highest groundwater level for the site ranged between 40 and 60 feet below the existing grade observed along Newhall Avenue. Based on the depth to the historically highest and current groundwater levels, as well as density of the site soils and bedrock, it is the opinion of this firm that the potential for liquefaction impacting the proposed development is considered negligible.

Dynamic Dry Settlement

Seismically-induced settlement or compaction of dry or moist, cohesionless soils can be an effect related to earthquake ground motion. Such settlements are typically most damaging when the settlements are differential in nature across the length of structures.

A site-specific seismic dry sand settlement analysis was performed utilizing Tokimatsu and Seed's procedure for the alluvial and older alluvial soils encountered in Boring B1 (Tokimatsu and Seed, 1987). The enclosed dynamic dry settlement analysis is based on a peak ground acceleration (PGA_M) of 1.175g, and a mean magnitude (M_W) of 6.7. These values were obtained from the SEAOC/OSHPD U.S. Seismic Design Maps tool and the USGS Probabilistic Seismic Hazard Deaggregation program (USGS, 20124). Since groundwater was no encountered, the enclosed dynamic dry settlement analysis was evaluated to a depth of 50 feet.

Based on the parameters provided above, the enclosed seismically-induced dry sand settlement calculation resulted in a total dynamic dry settlement of ½-inch. Differential dynamic dry settlement would not be expected to exceed two-thirds of the total dynamic settlement, or ⅓-inch.

Tsunamis, Seiches and Flooding

Tsunamis are large ocean waves generated by sudden water displacement caused by a submarine earthquake, landslide, or volcanic eruption. Review of the County of Los Angeles Flood and Inundation Hazards Map (Leighton, 1990) indicates the site does not lie within mapped tsunami inundation boundaries.

Review of the County of Los Angeles Flood and Inundation Hazards Map, (Leighton, 1990), indicates the northern portion of the site, along Newhall Avenue, lies within mapped inundation boundaries due to an upgradient reservoir. A determination of whether a higher site elevation would remove the site from the potential inundation zones is beyond the scope of this assessment.

A copy of the Flood Insurance Rate Map (FIRM) for the project site is obtained from the FEMA Flood Map Service Center website (https://msc.fema.gov/portal/search), and it is attached to this report. Based on review of the FIRM map, the majority of the site is located within an "Area of Minimal Flood Hazard" (Zone X). However, the northern portion of the site, along Newhall Avenue, is located within an area labeled "0.2 annual chance flood hazard, areas of 1 percent annual chance flood with average depth less than 1 foot or with drainage areas of less than 1 square mile".

Landsliding

According to the Earthquake Zones of Required Investigation Maps of the State of California (CDMG, 1998), a small portion of the western ridge is mapped to be within an "Earthquake Induced Landslide" zone. A copy of this map may be found in the Appendix. However, this ridge has been recently graded and trimmed down to approximate elevation 1,380 feet, as discussed above, for the creation of a level terrace at the neighboring site. Furthermore, the remaining 1½:1 slope will be redefined as part of the proposed development, and will be cut at a maximum 2:1 slope gradient. The existing and proposed slopes for this area are illustrated in the enclosed Cross Sections B-B' and C-C'. Based on the results from the slope stability analyses addressed in the following section of this report, it is the opinion of this firm that the potential for earthquake induced landslides at this area, and the rest of the site, may be considered remote.

Review of the County of Los Angeles Landslide Inventory Map (Leighton, 1990), the Geologic Map of the Oat Mountain and Canoga Park (North ½) Quadrangles (Dibblee, T.W., 1992), the Geologic Map of Part of the Ventura Basin (Winterer and Durham, 1962), and the Seismic Hazard Zone Report of the Oat Mountain Quadrangle, by the State of California, Department of Conservation, Division of Mines and Geology (CDMG SHZR 05, 1997, revised 2006), indicates that the site and adjacent descending slopes do not lie within the boundaries of any mapped landslides.

The results of slope stability analyses are presented in the "Slope Stability" section below.

SLOPE STABILITY ANALYSIS

SLOPES DESCRIPTION

The site consists of a canyon located along the north-south direction, with two ridges ascending to the south and west. Within the site limits, the highest elevation observed at the eastern ridge is 1,440 feet. The highest elevation observed at the western ridge is 1,380 feet. The elevation at the canyon found within the central portion of the site ranges from 1,317 feet at the northern corner to 1,353 at the south. The existing slope gradients range from approximately 1½:1 to 4:1 (horizontal:vertical).

Based on the site reconnaissance and review of available aerial photographs, the slopes did not exhibit indications of instability such as hummocky topography, ground surface tension cracks, or arcuate-shaped scarps. No seeps or springs were noted during the site reconnaissance.

The most critical slope from the standpoint of stability will occur at Cross Sections A-A', C-C', and D-D'. Therefore, the slope stability analysis was performed for these three cross sections.

ANALYSIS METHODOLOGY

The procedure for the stability analyses was performed in accordance with the screening procedures set forth in CGS Special Publication 117A (CDMG, 2008), (Blake and others, 2002), and (Stewart and others, 2003). The computer program SLIDE2 by RocScience was used for the analysis of Cross Sections A-A', C-C', and D-D' for analysis of the slopes. A discussion of the parameters used in the stability analyses is presented below.

Soil Strength

The strength of the geologic materials was determined by performing direct shear tests on the geologic materials at various normal loads. All of the samples were saturated prior to shearing. The geologic material properties are presented in the A and B Plates of the report Appendix. A summary of the material strengths used in the analysis is tabulated below.

Summary of Geologic Material Strengths Used in Stability Analyses				
Geologic Material	**Modeled Strength Characteristics**	**Moist Unit Weight (pcf)**	**Cohesion (psf)**	**Angle of Internal Friction (degrees)**
Compacted Fill	Isotropic	120	200	30
Alluvium	Isotropic	120	230	29
Older Alluvium	Isotropic	125	385	31
Bedrock (Saugus Formation)	AnIsotropic	120	650[1] 320[2]	35[1] 17.5[2]

Notes: [1]Denotes rock strength across bedding; [2] Denotes rock strength along bedding

Groundwater

Groundwater was not encountered in the exploratory excavations, which were conducted to a maximum depth of 50 feet below the existing grade. The historically-highest groundwater level for the site is mapped at depths between 40 and 60 feet below the lowest site grades (along Newhall Avenue). Based on these considerations, groundwater is not expected to affect the stability of the slopes, and has not been considered in the stability analyses.

External Loads

The existing slopes, as well as the upper and lower terraces, are undeveloped. A uniform load of 300 pounds per square feet has been assumed for the proposed townhome structures.

Seismic Load

The 2014 USGS Unified Hazard Tool was utilized to determine the deaggregation of the seismic risk for a 10% in 50 year probability of exceedance. A Site Class D was utilized, assuming a shear wave velocity of 360 m/s. The analyzed deaggregation resulted in an Earthquake Magnitude of 6.71, with a modal acceleration of 0.67g and a distance of 8.64 kilometers.

The enclosed calculation labeled "Estimation of Permanent Seismic Displacement using the Bray and Rathje (1998) Procedure" is based on the above earthquake acceleration and magnitude. Because the potential failure planes are not expected to extend into any of the proposed structures, a displacement value of 15 cm was utilized. A Seismic Coefficient for Screen Procedure (k_{eq}) of 0.234g was determined. This value was utilized in the slope stability calculations which are summarized in the following table.

Analysis Parameters

The stability analyses were performed using Bishop's and Janbu's methods to analyze both circular and block shaped failure surfaces. Five thousand searches for the lowest factor of safety were performed for the analyses.

Results

The stability analyses indicates that the existing slope with proposed improvements has a factor of safety in excess of the County of Los Angeles Department of Public Works minimum requirement of 1.5 for static conditions and 1.10 for pseudostatic conditions. The computer output files are included in the Appendix. The results are summarized below.

Summary of Stability Analyses - JANBU METHOD			
CROSS SECTION	**ANALYSIS TYPE**	**ANALYSIS CONDITION**	**FACTOR OF SAFETY**
A-A'	(circular)	Static	1.92 (1.5 min allowable)
	(circular)	Pseudostatic	1.18 (1.10 min. allowable)
	(block)	Static	1.96 (1.5 min. allowable)
	(block)	Pseudostatic	1.19 (1.10 min. allowable)
C-C'	(circular)	Static	2.09 (1.5 min allowable)
	(circular)	Pseudostatic	1.51 (1.10 min. allowable)
	(block)	Static	2.18 (1.5 min. allowable)
	(block)	Pseudostatic	1.44 (1.10 min. allowable)
D-D'	(circular)	Static	2.22 (1.5 min allowable)
	(circular)	Pseudostatic	1.38 (1.10 min. allowable)
	(block)	Static	2.41 (1.5 min. allowable)
	(block)	Pseudostatic	1.52 (1.10 min. allowable)

Summary of Stability Analyses - **BISHOP METHOD**			
CROSS SECTION	**ANALYSIS TYPE**	**ANALYSIS CONDITION**	**FACTOR OF SAFETY**
A-A'	(circular)	Static	1.96 (1.5 min allowable)
	(circular)	Pseudostatic	1.18 (1.10 min. allowable)
	(block)	Static	2.02 (1.5 min. allowable)
	(block)	Pseudostatic	1.25 (1.10 min. allowable)
C-C'	(circular)	Static	1.70 (1.5 min allowable)
	(circular)	Pseudostatic	1.49 (1.10 min. allowable)
	(block)	Static	2.22 (1.5 min. allowable)
	(block)	Pseudostatic	1.55 (1.10 min. allowable)
D-D'	(circular)	Static	2.31 (1.5 min allowable)
	(circular)	Pseudostatic	1.38 (1.10 min. allowable)
	(block)	Static	2.59 (1.5 min. allowable)
	(block)	Pseudostatic	1.65 (1.10 min. allowable)

Discussion

All of the slope analyses identified a static factor of safety exceeding the minimum required 1.5 and the pseudostatic minimum value of 1.10. The printouts of the slope stability calculations are provided in the Appendix of this report.

Surficial Stability

The slope proposed along Cross Sections A-A' and C-C' were checked for surficial stability. These slopes will be inclined at a 2:1 (horizontal to vertical) gradient. The analysis used was developed by Blake, Hollingsworth and Stewart (2002). The factors of safety for these slopes were 2.04 for A-A', and 4.09 for C-C'. These factors of safety are considered to be adequate.

CONCLUSIONS AND RECOMMENDATIONS

Based upon the exploration, laboratory testing, and research, it is the finding of Geotechnologies, Inc. that construction of the proposed development is considered feasible from a geotechnical engineering standpoint provided the advice and recommendations presented herein are followed and implemented during construction.

As illustrated in the enclosed Proposed Site Plan and cross sections, it is anticipated that grading of the site will consist of cutting of some of the existing slopes, and filling some of the low areas, for the creation of level terraces. Several retaining walls are proposed throughout the site to aid in the creation of the proposed level terraces. These retaining walls are anticipated to range from 4 to 18 feet in height.

The proposed cut-slopes will be up to 36 feet in height, and will be cut at a maximum slope gradient of 2:1 (horizontal:vertical). The cut slopes are expected to expose older alluvial soils on the eastern portion of the site, and older alluvial soils and neutrally oriented bedrock of the Saugus Formation on the western portion of the site. Based on results from the slope stability analyses, it is the opinion of this firm that the proposed 2:1 permanent slope cuts will be stable.

It is anticipated that the placement of up to 15 feet of fill soils will be required within some of the lower areas. Within these areas, it is recommended that the existing uncertified fill materials are removed prior to the placement of new fill. It is anticipated that the majority of this grading will be completed utilizing soils derived from the proposed cut slopes. Utilizing these materials to raise the grade at the low areas is acceptable. If the importation of soils will be necessary to complete the proposed grading, these soils shall be approved by the geotechnical engineer of record before they are transported to the site.

The proposed retaining walls may be supported by conventional foundations bearing in undisturbed alluvium, older alluvium, or bedrock. The existing uncertified fill materials are not suitable for foundation support. Where temporary embankments are necessary for the construction of new retaining walls, these embankments may be cut at a 1:1 gradient, up to a height of 25 feet.

The proposed residential structures and recreation center may be supported by conventional foundations. The foundations to support each individual structure should bear in the same geologic material. As illustrated in the enclosed cross sections, depending on their location, it is anticipated that the subgrade of the proposed level terraces will consist of recompacted soils, alluvium, older alluvium, and bedrock.

Where alluvium, older alluvium or bedrock is exposed at the subgrade of an individual structure, the foundations for this structure may bear in that particular material. Where different types of materials are exposed at the structure's footprint, it is recommended that earth materials should be removed, blended, and recompacted for the creation of a uniform compacted fill pad. For the creation of a compacted fill pad, earth materials should be removed and recompacted to a minimum depth of 4 feet below the proposed grade, or 2 feet below the bottom of the proposed foundations, whichever is deeper. In addition, the compacted fill should extend horizontally a minimum of 3 feet beyond the edge of foundations or for a distance equal to the depth of fill below the foundation, whichever is greater.

Where the subgrade of an individual structure will consist of recompacted soils placed to level, or raise an existing grade, it is recommended that the depth of the recompacted sections extends to a minimum depth of 4 feet below the proposed subgrade, or 2 feet below the bottom of the proposed foundations, whichever is deeper. In addition, the compacted fill should extend horizontally a minimum of 3 feet beyond the edge of foundations or for a distance equal to the depth of fill below the foundation, whichever is greater. If the compacted fill is placed in such way, it would be suitable for support of the proposed structure.

The validity of the conclusions and design recommendations presented herein is dependent upon review of the geotechnical aspects of the proposed construction by this firm. The subsurface conditions described herein have been projected from borings on the site as indicated and should in no way be construed to reflect any variations which may occur between these borings or which may result from changes in subsurface conditions. Any changes in the design or location of any structure, as outlined in this report, should be reviewed by this office. The recommendations contained herein should not be considered valid until reviewed and modified or reaffirmed subsequent to such review.

The following statement is made in regard to Los Angeles County Code Sections 110 and 111: It is the opinion of the undersigned based on the findings of this investigation that provided the recommendations presented in this report are followed, the proposed development will be safe for its intended use against hazard from landsliding, settlement or slippage. The proposed development will have no adverse effect on the stability of the site of adjoining properties.

SEISMIC DESIGN CONSIDERATIONS

California Building Code Seismic Parameters

Based on information derived from the subsurface investigation, the subject site is classified as Site Class D, which corresponds to a "Stiff Soil" Profile, according to Table 20.3-1 of ASCE 7-16. This information and the site coordinates were input into the OSHPD seismic utility program in order to calculate ground motion parameters for the site.

CALIFORNIA BUILDING CODE SEISMIC PARAMETERS	
California Building Code	2019
ASCE Design Standard	7-16
Site Class	D
Mapped Spectral Acceleration at Short Periods (S_S)	2.544g
Site Coefficient (F_a)	1.0
Maximum Considered Earthquake Spectral Response for Short Periods (S_{MS})	2.544g
Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	1.696g
Mapped Spectral Acceleration at One-Second Period (S_1)	0.824g
Site Coefficient (F_v)	1.7*
Maximum Considered Earthquake Spectral Response for One-Second Period (S_{M1})	1.400g*
Five-Percent Damped Design Spectral Response Acceleration for One-Second Period (S_{D1})	0.934g*

* *According to ASCE 7-16, a Long Period Site Coefficient (F_v) of 1.7 may be utilized provided that the value of the Seismic Response Coefficient (C_s) is determined by Equation 12.8-2 for values of $T \leq 1.5T_s$ and taken as equal to 1.5 times the value computed in accordance with either Equation 12.8-3 for $T_L \geq T > 1.5T_s$ or equation 12.8-4 for $T > T_L$. Alternatively, a site-specific ground motion hazard analysis may be performed in accordance with ASCE 7-16 Section 21.1 and/or a ground motion hazard analysis in accordance with ASCE 7-16 Section 21.2 to determine ground motions for any structure.*

EXPANSIVE SOILS

The onsite geologic materials are in the low to very low expansion range. The Expansion Index was found to be between 7 and 28 for a representative bulk samples. Recommended reinforcing is provided in the "Foundation Design" and "Slab-On-Grade" sections of this report.

SOIL CORROSION POTENTIAL

The results of the soil corrosivity testing performed on six samples representative of the onsite soils by Project X Corrosion Engineering indicate that the electrical resistivities of the soils are corrosive to general metals when saturated. The soil pH value of the samples was between 6.3 and 7.6. These pH levels are considered not detrimental to copper and aluminum alloys. Chloride levels in the samples are low and may cause insignificant corrosion of metals. Ammonia and Nitrates concentrations were high enough to cause accelerated corrosion of copper and copper alloys, such as brass.

Sulfate content in the samples are considered negligible for corrosion of cement. Special cement types need not be utilized for concrete structures in contact with the soils, since the sulfate content of the soils is negligible.

Detailed results, discussion of results and recommended mitigating measures are provided within the enclosed Corrosion Evaluation Report prepared by Project X Corrosion Engineering, dated February 11, 2022.

GRADING GUIDELINES

The following guidelines are provided for the preparation of compacted fill pads, and for the grading of areas where the current grade will be raised. Fill slopes are not anticipated as part of the proposed development; therefore hillside grading recommendations are not provided.

Site Preparation

- A thorough search should be made for possible underground utilities and/or structures. Any existing or abandoned utilities or structures located within the footprint of the proposed grading should be removed or relocated as appropriate.

- All vegetation, existing fill, and soft or disturbed geologic materials should be removed from the areas to receive controlled fill. All existing fill materials and any disturbed geologic materials resulting from grading operations shall be completely removed and properly recompacted prior to foundation excavation.

- Any vegetation or associated root system located within the footprint of the proposed structures should be removed during grading.

- Subsequent to the indicated removals, the exposed grade shall be scarified to a depth of six inches, moistened to optimum moisture content, and recompacted in excess of the minimum required comparative density.

- The excavated areas shall be observed by the geotechnical engineer prior to placing compacted fill.

Recommended Overexcavation and Blending

In areas were different geologic materials will be exposed at the subgrade of an individual structure, it is recommended that these materials are removed, blended, and recompacted for the creation of a uniform compacted fill pad. These proposed building areas shall be excavated to a minimum depth of 4 feet below the proposed subgrade, or 2 feet below the bottom of the proposed foundations, whichever is greater. The excavation shall extend at least three feet beyond the edge of foundations or for a distance equal to the depth of fill below the foundations, whichever is greater.

Similarly, in areas where the proposed grade will be raised prior to construction of a structure, it is recommended that the thickness of the recompacted fill extends to a minimum depth of 4 feet below the proposed subgrade, or 2 feet below the bottom of the proposed foundations, whichever is greater. The excavation shall extend at least three feet beyond the edge of foundations or for a distance equal to the depth of fill below the foundations, whichever is greater.

It is very important that the positions of the proposed structures are accurately located so that the limits of the graded area are accurate and the grading operation proceeds efficiently.

Compaction

All fill should be mechanically compacted in layers not more than 8 inches thick. The materials placed should be moisture conditions to within 3 percent of the optimum moisture content of the particular material placed. All fill shall be compacted to at least 90 percent of the maximum laboratory density for the materials used. The maximum density shall be determined by the laboratory operated by Geotechnologies, Inc. in general accordance with the most recent revision of ASTM D 1557.

Field observation and testing shall be performed by a representative of the geotechnical engineer during grading to assist the contractor in obtaining the required degree of compaction and the proper moisture content. Where compaction is less than required, additional compactive effort shall be made with adjustment of the moisture content, as necessary, until a minimum of 90 percent compaction is obtained.

Acceptable Materials

The excavated onsite geologic materials are considered satisfactory for reuse in compacted fills as long as any oversize material, debris and/or organic matter is removed. Cobbles should be expected to be present in the on-site materials. Cobbles exceeding 6 inches in dimension shall not be utilized in compacted fills.

Any imported soil shall be observed and tested by the representative of the geotechnical engineer prior to use in fill areas. Imported materials should contain sufficient fines so as to be relatively impermeable and result in a stable subgrade when compacted. Any required import soil should have an expansion index less than 40. The water-soluble sulfate content of the import materials should be less than 0.1% percentage by weight.

Imported soil should be free from chemical or organic substances which could affect the proposed development. A competent professional should be retained in order to test imported materials and address environmental issues and organic substances which might affect the proposed development.

Utility Trench Backfill

Utility trenches should be backfilled with controlled fill. The utility should be bedded with clean sands at least one foot over the crown. The remainder of the backfill may be onsite soil compacted to 90 percent of the laboratory maximum density. Utility trench backfill should be tested by representatives of this firm in general accordance with the most recent revision of ASTM D 1557.

Shrinkage

Shrinkage results when a volume of soil removed at one density is compacted to a higher density. A shrinkage factor between 10 and 20 percent should be anticipated when excavating and recompacting the existing fill and underlying native geologic materials on the site to an average comparative compaction of 92 percent.

Weather Related Grading Considerations

When rain is forecast all fill that has been spread and awaits compaction shall be properly compacted prior to stopping work for the day or prior to stopping due to inclement weather. These fills, once compacted, shall have the surface sloped to drain to an area where water can be removed.

Temporary drainage devices should be installed to collect and transfer excess water to the street in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. Drainage should not be allowed to flow uncontrolled over any descending slope.

Work may start again, after a period of rainfall, once the site has been reviewed by a representative of this office. Any soils saturated by the rain shall be removed and aerated so that the moisture content will fall within three percent of the optimum moisture content.

Surface materials previously compacted before the rain shall be scarified, brought to the proper moisture content and recompacted prior to placing additional fill, if considered necessary by a representative of this firm.

Abandoned Seepage Pits

No abandoned seepage pits were encountered during exploration and none are known to exist on the site. However, should such a structure be encountered during grading, options to permanently abandon seepage pits include complete removal and backfill of the excavation with compacted fill, or drilling out the loose materials and backfilling to within a few feet of grade with slurry, followed by a compacted fill cap.

If the subsurface structures are to be removed by grading, the entire structure should be demolished. The resulting void may be refilled with compacted soil. Concrete and brick generated during the seepage pit removal may be reused in the fill as long as all fragments are less than 2 inches in longest dimension and the debris comprises less than 20 percent of the fill by volume. All grading should comply with the recommendations of this report.

Where the seepage pit structure is to be left in place, the seepage pits should be cleaned of all soil and debris. This may be accomplished by drilling. The pits should be filled with minimum 1½-sack concrete slurry to within 5 feet of the bottom of the proposed foundations. In order to provide a more uniform foundation condition, the remainder of the void should be filled with controlled fill.

Geotechnical Observations and Testing During Grading

Geotechnical observations and testing during grading are considered to be a continuation of the geotechnical investigation. It is critical that the geotechnical aspects of the project be reviewed by representatives of Geotechnologies, Inc. during the construction process. Compliance with the design concepts, specifications or recommendations during construction requires review by this firm during the course of construction. Any fill which is placed should be observed, tested, and verified if used for engineered purposes. Please advise this office at least twenty-four hours prior to any required site visit.

Proper compaction is necessary to reduce settlement of overlying improvements. Some settlement of compacted fill should be anticipated. Any utilities supported therein should be designed to accept differential settlement. Differential settlement should also be considered at the points of entry to the structure.

LEED Considerations

The Leadership in Energy and Environmental Design (LEED) Green Building Rating System encourages adoption of sustainable green building and development practices. Credit for LEED Certification can be assigned for reuse of construction waste and diversion of materials from landfills in new construction.

In an effort to provide the design team with a viable option in this regard, demolition debris could be crushed onsite in order to use it in the ongoing grading operations. The environmental ramifications of this option, if any, should be considered by the team.

The demolition debris should be limited to concrete, asphalt and other non-deleterious materials. All deleterious materials should be removed including, but not limited to, paper, garbage, ceramic materials and wood.

For structural fill applications, the materials should be crushed to 2 inches in maximum dimension or smaller. The crushed materials should be thoroughly blended and mixed with onsite soils prior to placement as compacted fill. The amount of crushed material should not exceed 20 percent. The blended and mixed materials should be tested by this office prior to placement to insure it is suitable for compaction purposes. The blended and mixed materials should be tested by Geotechnologies, Inc. during placement to ensure that it has been compacted in a suitable manner.

FOUNDATION DESIGN

Conventional

The proposed residential structures, recreation center and miscellaneous retaining walls may be supported by conventional foundations. Depending on their location, conventional foundations to support the proposed residential structures and recreation center may bear in a newly placed compacted fill pad, alluvium, older alluvium, or bedrock. All the foundations for an individual structure shall bear in the same type of material.

Conventional foundations to support the proposed retaining walls may bear in alluvium, older alluvium and bedrock. Foundations for an individual retaining wall may transition across different materials.

All continuous foundations required for the project may be designed for a bearing capacity of 3,000 pounds per square foot, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material.

All column foundations required for the project may be designed for a bearing capacity of 3,500 pounds per square foot, and should be a minimum of 24 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material.

The bearing capacity increase for each additional foot of width is 250 pounds per square foot. The bearing capacity increase for each additional foot of depth is 500 pounds per square foot. The maximum recommended bearing capacity is 5,000 pounds per square foot.

The bearing capacities indicated above are for the total of dead and frequently applied live loads, and may be increased by one third for short duration loading, which includes the effects of wind or seismic forces.

Miscellaneous Conventional Foundations

Conventional foundations for structures such as carports, privacy walls and trash enclosures, which will not be rigidly connected to the proposed structures, may bear in native alluvial soils, older alluvial soils, bedrock, and properly compacted fill. Continuous and column footings may be designed for a bearing capacity of 2,000 pounds per square foot, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material. No bearing capacity increases are recommended.

Since the recommended bearing capacity is a net value, the weight of concrete in the foundations may be taken as 50 pounds per cubic foot and the weight of the soil backfill may be neglected when determining the downward load on the foundations.

Foundation Reinforcement

All continuous foundations should be reinforced with a minimum of four #4 steel bars. Two should be placed near the top of the foundation, and two should be placed near the bottom.

Lateral Design

Resistance to lateral loading may be provided by friction acting at the base of foundations and by passive earth pressure. An allowable coefficient of friction of 0.38 may be used with the dead load forces.

Passive geologic pressure for the sides of foundations poured against undisturbed native materials or recompacted fill may be computed as an equivalent fluid having a density of 250 pounds per cubic foot with a maximum earth pressure of 2,000 pounds per square foot.

The passive and friction components may be combined for lateral resistance without reduction. A one-third increase in the passive value may be used for short duration loading such as wind or seismic forces.

Foundation Settlement

Settlement of the foundation system is expected to occur on initial application of loading. The maximum static settlement is not expected to exceed ½-inch and occur below the heaviest loaded columns. Differential static settlement between the new foundations is not expected to exceed ¼-inch.

In addition to static settlement, the foundation system shall be designed to withstand dynamic (seismic) settlement by compaction of dry materials. As presented in a previous section of this report, the maximum total seismic settlement due to a major seismic event is expected to be on the order of ½-inch, and the anticipated seismically induced differential settlement is anticipated to be on the order of ⅓- inch. The static and seismic settlement reported herein are additive. The differential settlement would occur over a distance of 30 feet.

POOL SHELL DESIGN

It is recommended that the proposed pool is supported on a uniform compacted fill pad, which extends to a depth of 2 feet below the bottom of the pool. Additionally, the compacted fill pad shall extend horizontally 3 feet beyond the edge of the pool. A subdrain and/or a hydrostatic relief value are recommended. Exterior pool walls, up to 10 feet in height, should be designed to resist a triangular equivalent fluid pressure of 30 pcf.

RETAINING WALL DESIGN

Miscellaneous cantilever retaining walls are expected throughout the site, to aid in the creation of the proposed level terraces. These walls are expected to range from 4 to 18 feet in height, and will retain geologic materials consisting of recompacted fill, alluvial soils, older alluvial soils and neutrally bedded bedrock. The height of each individual wall is provided in the enclosed Proposed Site Plan.

The majority of the proposed retaining wall will have an ascending slope at their top. This slope is not expected to exceed a 2:1 slope gradient. Very few retaining walls will have a level backslope. Retaining walls may be designed utilizing the following table:

Height of Wall (Feet)	Walls with <u>Ascending</u> Backslope Triangular Distribution of Pressure (Pounds per Cubic Foot)	Walls with <u>Level</u> Backslope Triangular Distribution of Pressure (Pounds per Cubic Foot)
Up to 11 feet	43	30
11 to 14	43	34
14 to 18	56	39
18 to 20	62	40

For this equivalent fluid pressure to be valid, walls which are to be restrained at the top should be backfilled prior to the upper connection being made. Additional active pressure should be added for a surcharge condition due to vehicular traffic or adjacent structures.

All walls retaining an ascending slope should maintain a minimum of 2 feet of freeboard. In addition, a concrete swale shall be provided behind the proposed retaining walls to aid in facilitating drainage. Drainage shall be collected and discharged to an acceptable drainage area.

Dynamic (Seismic) Earth Pressure

Retaining walls exceeding 6 feet in height shall be designed to resist the additional earth pressure caused by seismic ground shaking. The seismic earth pressure was analyzed based on Memo S004.0, revised January 6, 2020, provided in the Administrative Manual of the County of Los Angeles, Department of Public Works. The attached spreadsheet titled "Seismically Induced Lateral Soil Pressure on Retaining Wall" shows the calculations. The input value is based on the Peak Ground Acceleration, which corresponds to the Short Term Design Acceleration divided by 2.5. The S_{DS} value was derived from the OSHPD seismic utility program. Results are summarized below:

RETAINING WALL CONDITION	DYNAMIC EARTH PRESSURE (Equivalent Fluid Pressure)
Cantilever (Unrestrained) Walls with Level Backfill	34 pcf
Cantilever (Unrestrained) Walls with Sloping Backfill	57 pcf

The earth pressure distribution is triangular in shape with the force applied at a height of 0.37H from the base of the wall, where H is the height of the wall.

Retaining Wall Drainage

All retaining walls shall be provided with a subdrain system in order to minimize the potential for future hydrostatic pressure buildup behind the proposed retaining walls. Subdrains may consist of 4-inch diameter perforated pipes, places with perforated facing down. The pipe shall be encased in at least one foot of gravel around the pipe. The gravel shall be wrapped in filter fabric. The gravel may consist of three-quarter inch to one-inch crushed rock.

As an alternative, the use of gravel pockets and weepholes is an acceptable drainage method. Weepholes shall be a minimum of 2 inches in diameter, placed at 8 feet on center along the base of the wall. Gravel pockets shall be a minimum of 1 cubic foot in dimension, and may consist of three-quarter inch to once inch crushed rock, wrapped in filter fabric.

Certain types of subdrain pipe are not acceptable to the various municipal agencies, it is recommended that prior to purchasing subdrainage pipe, the type and brand is cleared with the proper municipal agencies. Subdrainage pipes should outlet to an acceptable location.

The lateral earth pressures recommended above for retaining walls assume that a permanent drainage system will be installed so that external water pressure will not be developed against the walls. If a drainage system is not provided, the walls should be designed to resist an external hydrostatic pressure due to water in addition to the lateral earth pressure. In any event, it is recommended that retaining walls be waterproofed.

Waterproofing

Moisture effecting retaining walls is one of the most common post construction complaints. Poorly applied or omitted waterproofing can lead to efflorescence or standing water inside the building. Efflorescence is a process in which a powdery substance is produced on the surface of the concrete by the evaporation of water. The white powder usually consists of soluble salts such as gypsum, calcite, or common salt. Efflorescence is common to retaining walls and does not affect their strength or integrity.

Waterproofing is recommended for retaining walls. Waterproofing design and inspection of its installation is not the responsibility of the geotechnical engineer. A qualified waterproofing consultant should be retained in order to recommend a product or method which would provide protection to below grade walls.

Retaining Wall Backfill

Any required backfill should be mechanically compacted in layers not more than 8 inches thick, to at least 90 percent of the maximum density in general accordance with the most recent revision of ASTM D 1557 method of compaction. Flooding should not be permitted. Compaction within 5 feet, measured horizontally, behind a retaining structure should be achieved by use of light weight, hand operated compaction equipment.

Proper compaction of the backfill will be necessary to reduce settlement of overlying walks and paving. Some settlement of required backfill should be anticipated, and any utilities supported therein should be designed to accept differential settlement, particularly at the points of entry to the structure.

TEMPORARY EXCAVATIONS

The on-site geologic materials are suitable for vertical excavations up to 5 feet where not surcharged by adjacent traffic, structures or property lines.

Higher temporary excavations are anticipated for construction of the taller retaining walls. These excavations may be performed with the aid of temporary embankments. These temporary unsurcharged embankments could be cut at a uniform 1:1 slope gradient to a maximum height of 25 feet, and at a 1½:1 to a maximum height of 45 feet. A uniform sloped excavation is sloped from bottom to top and does not have a vertical component.

Where sloped embankments are utilized, the tops of the slopes should be barricaded to prevent vehicles and storage loads near the top of slope within a horizontal distance equal to the depth of the excavation. If the temporary construction embankments are to be maintained during the rainy season, berms are strongly recommended along the tops of the slopes to prevent runoff water from entering the excavation and eroding the slope faces. Water should not be allowed to pond on top of the excavation nor to flow towards it.

Excavation Observations

It is critical that the soils exposed in the cut slopes are observed by a representative of Geotechnologies, Inc. during excavation so that modifications of the slopes can be made if variations in the geologic material conditions occur. Many building officials require that temporary excavations should be made during the continuous observations of the geotechnical engineer. All excavations should be stabilized within 30 days of initial excavation.

SLABS ON GRADE

Concrete Slabs-on Grade

Concrete floor slabs and outdoor concrete flatwork should be a minimum of 4 inches in thickness. Slabs-on-grade and outdoor concrete flatwork should be cast over undisturbed natural geologic materials or properly controlled fill materials. Any geologic materials loosened or over-excavated should be wasted from the site or properly compacted to 90 percent of the maximum dry density.

Design of Slabs That Receive Moisture-Sensitive Floor Coverings

Geotechnologies, Inc. does not practice in the field of moisture vapor transmission evaluation and mitigation. Therefore, it is recommended that a qualified consultant be engaged to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. The qualified consultant should provide recommendations for mitigation of potential adverse impacts of moisture vapor transmission on various components of the structure.

Where dampness would be objectionable, it is recommended that the floor slabs should be waterproofed. A qualified waterproofing consultant should be retained in order to recommend a product or method which would provide protection for concrete slabs-on-grade.

All concrete slabs-on-grade should be supported on vapor retarder. The design of the slab and the installation of the vapor retarder should comply with the most recent revisions of ASTM E 1643 and ASTM E 1745. The vapor retarder should comply with ASTM E 1745 Class A requirements.

Where a vapor retarder is used, a low-slump concrete should be used to minimize possible curling of the slabs. The barrier can be covered with a layer of trimable, compactible, granular fill, where it is thought to be beneficial. See ACI 302.2R-32, Chapter 7 for information on the placement of vapor retarders and the use of a fill layer.

Concrete Crack Control

The recommendations presented in this report are intended to reduce the potential for cracking of concrete slabs-on-grade due to settlement. However even where these recommendations have been implemented, foundations, stucco walls and concrete slabs-on-grade may display some cracking due to minor soil movement and/or concrete shrinkage. The occurrence of concrete cracking may be reduced and/or controlled by limiting the slump of the concrete used, proper concrete placement and curing, and by placement of crack control joints at reasonable intervals, in particular, where re-entrant slab corners occur.

For standard control of concrete cracking, a maximum crack control joint spacing of 15 feet should not be exceeded. Lesser spacings would provide greater crack control. Joints at curves and angle points are recommended. The crack control joints should be installed as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. Construction joints should be designed by a structural engineer.

Complete removal of the existing fill soils beneath outdoor flatwork such as walkways or patio areas, is not required, however, due to the rigid nature of concrete, some cracking, a shorter design life and increased maintenance costs should be anticipated. In order to provide uniform support beneath the flatwork it is recommended that a minimum of 12 inches of the exposed subgrade beneath the flatwork be scarified and recompacted to 90 percent relative compaction.

Slab Reinforcing

Concrete slabs-on-grade should be reinforced with a minimum of #3 steel bars on 18-inch centers each way. Outdoor flatwork should be reinforced with a minimum of #3 steel bars on 24-inch centers each way.

PAVEMENTS

Prior to placing paving, the existing grade should be scarified to a depth of 12 inches, moistened as required to obtain optimum moisture content, and recompacted to 95 relative compaction, as determined by the most recent revision of ASTM D 1557. The client should be aware that removal of all existing fill in the area of new paving is not required, however, pavement constructed in this manner will most likely have a shorter design life and increased maintenance costs. The following pavement sections are recommended:

Service	Asphalt Pavement Thickness Inches	Base Course Inches
Passenger Cars	3	4
Moderate Truck	4	6
Heavy Truck	5	8

Aggregate base should be compacted to a minimum of 95 percent of the most recent revision of ASTM D 1557 laboratory maximum dry density. Base materials should conform to Sections 200-2.2 or 200-2.4 of the "Standard Specifications for Public Works Construction", (Green Book), latest edition.

Concrete paving may also be utilized for the project. For concrete paving sections to be subject to passenger cars and medium truck traffic, concrete paving shall be a minimum of 6 inches in thickness, and shall be underlain by 4 inches of aggregate base. For heavy truck traffic, concrete paving shall be a minimum of 7½ inches in thickness, and shall be underlain by 4 inches of aggregate base. For standard crack control maximum expansion joint spacing of 15 feet should not be exceeded. Lesser spacings would provide greater crack control. Joints at curves and angle points are recommended. Concrete paving should be reinforced with a minimum of #3 steel bars on 24-inch centers each way.

The performance of pavement is highly dependent upon providing positive surface drainage away from the edges. Ponding of water on or adjacent to pavement can result in saturation of the subgrade materials and subsequent pavement distress.

SITE DRAINAGE

Proper surface drainage is critical to the future performance of the project. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the designed engineering properties. Proper site drainage should be maintained at all times.

All site drainage, with the exception of any required to disposed of onsite by stormwater regulations, should be collected and transferred to the street in non-erosive drainage devices. The proposed structures should be provided with roof drainage. Discharge from downspouts, roof drains and scuppers should not be permitted on unprotected soils within five feet of the building perimeter. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. Drainage should not be allowed to flow uncontrolled over any descending slope. Planters which are located within a distance equal to the depth of a retaining wall should be sealed to prevent moisture adversely affecting the wall. Planters which are located within five feet of a foundation should be sealed to prevent moisture affecting the earth materials supporting the foundation.

STORMWATER DISPOSAL

Introduction

Recently regulatory agencies have been requiring the disposal of a certain amount of stormwater generated on a site by infiltration into the site soils. Increasing the moisture content of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the designed engineering properties. This means that any overlying structure, including buildings, pavements and concrete flatwork, could sustain damage due to saturation of the subgrade soils. Structures serviced by subterranean levels could be adversely impacted by stormwater disposal by increasing the design fluid pressures on retaining walls and causing leaks in the walls. Proper site drainage is critical to the performance of any structure in the built environment.

Percolation Testing

In order to determine the feasibility of on-site stormwater infiltration, percolation testing was conducted in Borings B1 and B2, following the procedure for boring percolation test provided in the Guidelines for Design, Investigation and Reporting Low Impact Development Stormwater Infiltration (GS200.1), dated June 30, 2021, presented in the Administrative Manual for the County of Los Angeles, Department of Public Works, Geotechnical and Material Engineering Division.

The location of Borings B1 and B2 is shown on the enclosed Geologic Map. These borings were drilled to a depth of 20 and 50 feet below the existing grade, with the aid of a truck-mounted drilling machine using 8-inch diameter hollowstem augers. At the completion of drilling, a 2-inch diameter casing was placed within the center of the borehole for the purpose of conducting percolation testing. The casing consisted of a slotted PVC pipe within the lower 30 feet for B1, and within the lower 10 feet for B2. A solid PVC pipe was installed to the top of the borehole. A sand pack consisting of #2 Monterey Sand was poured into the annular space around the slotted portion of the casing. A 1-foot thick, hydrated bentonite seal was placed over the sand and drill cuttings were placed to the ground surface.

Prior to testing, the boreholes were filled with water for the purpose of pre-soaking for 2 hours. After presoaking, the boreholes were refilled with water, and the rate of drop in the water level was measured. The percolation test readings were recorded a minimum of 8 times or until a stabilized rate of drop was obtained, whichever occurred first.

The table below summarizes the results of the infiltration rate derived from the testing. This rate includes correction factors (RF_t, RF_v, and RF_s), as required by the County of Los Angeles procedure. Field readings and calculations for the percolation testing are included in the Appendix.

Boring No.	Depth of Boring Below Existing Ground Surface (ft.)	Percolation Testing Conducted Between Depths (ft.)	Infiltration Rate (in./hr.)
B1	50	20 to 50	8.65
B2	20	10 to 20	7.09

Recommendations

Based on the results of the exploration, testing and research, it is the finding of this firm that on-site stormwater infiltration is feasible from a geotechnical standpoint. The design and location of any potential infiltration system has not been specifically addressed. Once a type of infiltration system, as well as its location, has been selected, this should be reviewed by this office to determine if supplemental recommendations are needed.

Stormwater infiltration shall only occur in native alluvial soils, or older alluvial soils. It is recommended that stormwater infiltration occurs below a depth of 10 feet below the proposed grade. The edge of any potential infiltration system shall maintain a minimum horizontal setback distance of 15 feet from any structure and private property line. Due to the granular nature of the underlying native alluvial soils, the stormwater should percolate in a generally vertical manner. The potential for creating a perched water condition is considered to be remote. The proposed stormwater infiltration system should not cause any damage, settlement, or adversely affect any neighboring buildings.

Soils located within the upper 10 feet strata shall not become wet or saturated as a result of on-site infiltration. State regulations require that the bottom of infiltration units maintain a minimum vertical distance of 10 feet above the groundwater level. Groundwater was not encountered to a depth of 50 feet below grade. Therefore, the bottom of any infiltration system should not extend below a depth of 40 feet below the existing grade.

The subject site is not located in an area considered susceptible to liquefaction. The onsite soils are in the very low to low expansion range, and are not susceptible to significant hydroconsolidation.

It is recommended that the design team, including the structural engineer, waterproofing consultant, plumbing engineer, environmental engineer and landscape architect be consulted in regard to the design and construction of infiltration systems. The design and construction of stormwater infiltration systems is not the responsibility of the geotechnical engineer. However, based on the experience of this firm, it is recommended that several aspects of the use of such facilities should be considered by the design and construction team:

- All infiltration devices should be provided with overflow protection. Once the device is full of water, additional water flowing to the device should be diverted to another acceptable disposal area or disposed offsite in an acceptable manner.

- All connections associated with stormwater infiltration devices should be sealed and water-tight. Water leaking into the subgrade soils can lead to loss of strength, piping, erosion, settlement and/or expansion of the effected earth materials.

- Excavations proposed for the installation of stormwater facilities should comply with the "Temporary Excavations" sections of this report as well as CalOSHA Regulations where applicable.

- Caving should be expected during drilling of the drywell. Where caving occurs, it will be necessary to utilize casing to maintain an open shaft. Cobbles and gravel should be expected during drilling.

DESIGN REVIEW

Engineering of the proposed project should not begin until approval of the geotechnical report by the Building Official is obtained in writing. Significant changes in the geotechnical recommendations may result during the building department review process.

It is recommended that the geotechnical aspects of the project be reviewed by this firm during the design process. This review provides assistance to the design team by providing specific recommendations for particular cases, as well as review of the proposed construction to evaluate whether the intent of the recommendations presented herein are satisfied.

CONSTRUCTION MONITORING

Geotechnical observations and testing during construction are considered to be a continuation of the geotechnical investigation. It is critical that this firm review the geotechnical aspects of the project during the construction process. Compliance with the design concepts, specifications or recommendations during construction requires review by this firm during the course of construction. All foundations should be observed by a representative of this firm prior to placing concrete or steel. Any fill which is placed should be observed, tested, and verified if used for engineered purposes. Please advise Geotechnologies, Inc. at least twenty-four hours prior to any required site visit.

If conditions encountered during construction appear to differ from those disclosed herein, notify Geotechnologies, Inc. immediately so the need for modifications may be considered in a timely manner.

It is the responsibility of the contractor to ensure that all excavations and trenches are properly sloped or shored. All temporary excavations should be cut and maintained in accordance with applicable OSHA rules and regulations.

EXCAVATION CHARACTERISTICS

The exploration performed for this investigation is limited to the geotechnical excavations described. Direct exploration of the entire site would not be economically feasible. The owner, design team and contractor must understand that differing excavation and drilling conditions may be encountered based on boulders, gravel, oversize materials, groundwater and many other conditions. Fill materials, especially when they were placed without benefit of modern grading codes, regularly contain materials which could impede efficient grading and drilling. Southern California sedimentary bedrock is known to contain variable layers which reflect differences in depositional environment. Such layers may include abundant gravel, cobbles and boulders. Similarly bedrock can contain concretions. Concretions are typically lenticular and follow the bedding. They are formed by mineral deposits. Concretions can be very hard. Excavation and drilling in these areas may require full size equipment and coring capability. The contractor should be familiar with the site and the geologic materials in the vicinity.

CLOSURE AND LIMITATIONS

The purpose of this report is to aid in the design and completion of the described project. Implementation of the advice presented in this report is intended to reduce certain risks associated with construction projects. The professional opinions and geotechnical advice contained in this report are sought because of special skill in engineering and geology and were prepared in accordance with generally accepted geotechnical engineering practice. Geotechnologies, Inc. has a duty to exercise the ordinary skill and competence of members of the engineering profession. Those who hire Geotechnologies, Inc. are not justified in expecting infallibility, but can expect reasonable professional care and competence.

The recommendations of this report pertain only to the site investigated and are based upon the assumption that the geologic conditions do not deviate from those disclosed in the investigation. If any variations are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geotechnologies, Inc. should be notified so that supplemental recommendations can be prepared.

This report is issued with the understanding that it is the responsibility of the owner, or the owner's representatives, to ensure that the information and recommendations contained herein are brought to the attention of the project architect and engineer and are incorporated into the plans. The owner is also responsible to see that the contractor and subcontractors carry out the geotechnical recommendations during construction.

The findings of this report are valid as of the date of this report. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may 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 control of this firm. Therefore, this report is subject to review and should not be relied upon after a period of three years.

Geotechnical observations and testing during construction is considered to be a continuation of the geotechnical investigation. It is, therefore, most prudent to employ the consultant performing the initial investigative work to provide observation and testing services during construction. This practice enables the project to flow smoothly from the planning stages through to completion.

Should another geotechnical firm be selected to provide the testing and observation services during construction, that firm should prepare a letter indicating their assumption of the responsibilities of geotechnical engineer of record. A copy of the letter should be provided to the regulatory agency for review. The letter should acknowledge the concurrence of the new geotechnical engineer with the recommendations presented in this report.

EXCLUSIONS

Geotechnologies, Inc. does not practice in the fields of methane gas, radon gas, environmental engineering, waterproofing, dewatering organic substances or the presence of corrosive soils or wetlands which could affect the proposed development including mold and toxic mold. Nothing in this report is intended to address these issues and/or their potential effect on the proposed development. A competent professional consultant should be retained in order to address environmental issues, waterproofing, organic substances and wetlands which might effect the proposed development.

GEOTECHNICAL TESTING

Classification and Sampling

The soil is continuously logged by a representative of this firm and classified by visual examination in accordance with the Unified Soil Classification system. The field classification is verified in the laboratory, also in accordance with the Unified Soil Classification System. Laboratory classification may include visual examination, Atterberg Limit Tests and grain size distribution. The final classification is shown on the excavation logs.

Samples of the geologic materials encountered in the exploratory excavations were collected and transported to the laboratory. Undisturbed samples of soil are obtained at frequent intervals. Samples from the hollow-stem auger drill rig are obtained by driving a thin-walled, California Modified Sampler with successive 30-inch drops of a 140-pound automatic-trip hammer. Samples from bucket-auger drilling are obtained utilizing a California Modified Sampler with successive 12-inch drops of a kelly bar, whose weight is noted on the excavation logs. Samples obtained from the test pits are obtained using a hand sampler. The soil is retained in brass rings of 2.50 inches outside diameter and 1.00 inch in height. The central portion of the samples are stored in close fitting, waterproof containers for transportation to the laboratory. Samples noted on the excavation logs as SPT samples are obtained in general accordance with the most recent revision of ASTM D 1586. Samples are retained for 30 days after the date of the geotechnical report.

Moisture and Density Relationships

The field moisture content and dry unit weight are determined for each of the undisturbed soil samples, and the moisture content is determined for SPT samples in general accordance with the most recent revision of ASTM D 4959 or ASTM D 4643. This information is useful in providing a gross picture of the soil consistency between exploration locations and any local variations. The dry unit weight is determined in pounds per cubic foot and shown on the "Excavation Logs", A-Plates. The field moisture content is determined as a percentage of the dry unit weight.

Direct Shear Testing

Shear tests are performed in general accordance with the most recent revision of ASTM D 3080 with a strain controlled, direct shear machine manufactured by Soil Test, Inc. or a Direct Shear Apparatus manufactured by GeoMatic, Inc. The rate of deformation ranges between approximately 0.005 and 0.025 inches per minute. Each sample is sheared under varying confining pressures in order to determine the Mohr-Coulomb shear strength parameters of the cohesion intercept and the angle of internal friction. Samples are generally tested in an artificially saturated condition. Depending upon the sample location and future site conditions, samples may be tested at field moisture content. The results are plotted on the "Shear Test Diagram," B-Plates.

The most recent revision of ASTM 3080 limits the particle size to 10 percent of the diameter of the direct shear test specimen. The sheared sample is inspected by the laboratory technician running the test. The inspection is performed by splitting the sample along the sheared plane and observing the soils exposed on both sides. Where oversize particles are observed in the shear plane, the results are discarded and the test run again with a fresh sample.

Consolidation Testing

Settlement predictions of the soil's behavior under load are made on the basis of the consolidation tests in general accordance with the most recent revision of ASTM D 2435. The consolidation apparatus is designed to receive a single one-inch high ring. Loads are applied in several increments in a geometric progression, and the resulting deformations are recorded at selected time intervals. Porous stones are placed in contact with the top and bottom of each specimen to permit addition and release of pore fluid. Samples are generally tested at increased moisture content to determine the effects of water on the bearing soil. The normal pressure at which the water is added is noted on the drawing. Results are plotted on the "Consolidation Test," C-Plates.

Expansion Index Testing

The expansion tests performed on the remolded samples are in accordance with the Expansion Index testing procedures, as described in the most recent revision of ASTM D 4829. The soil sample is compacted into a metal ring at a saturation degree of 50 percent. The ring sample is then placed in a consolidometer, under a vertical confining pressure of 1 lbf/square inch and inundated with distilled water. The deformation of the specimen is recorded for a period of 24 hour or until the rate of deformation becomes less than 0.0002 inches/hour, whichever occurs first. The expansion index, EI, is determined by dividing the difference between final and initial height of the ring sample by the initial height, and multiplied by 1,000. Results are presented in Plate D of this report.

Laboratory Compaction Characteristics

The maximum dry unit weight and optimum moisture content of a soil are determined in general accordance with the most recent revision of ASTM D 1557. A soil at a selected moisture content is placed in five layers into a mold of given dimensions, with each layer compacted by 25 blows of a 10 pound hammer dropped from a distance of 18 inches subjecting the soil to a total compactive effort of about 56,000 pounds per cubic foot. The resulting dry unit weight is determined. The procedure is repeated for a sufficient number of moisture contents to establish a relationship between the dry unit weight and the water content of the soil. The data when plotted represent a curvilinear relationship known as the compaction curve. The values of optimum moisture content and modified maximum dry unit weight are determined from the compaction curve. Results are presented in Plate D of this report.

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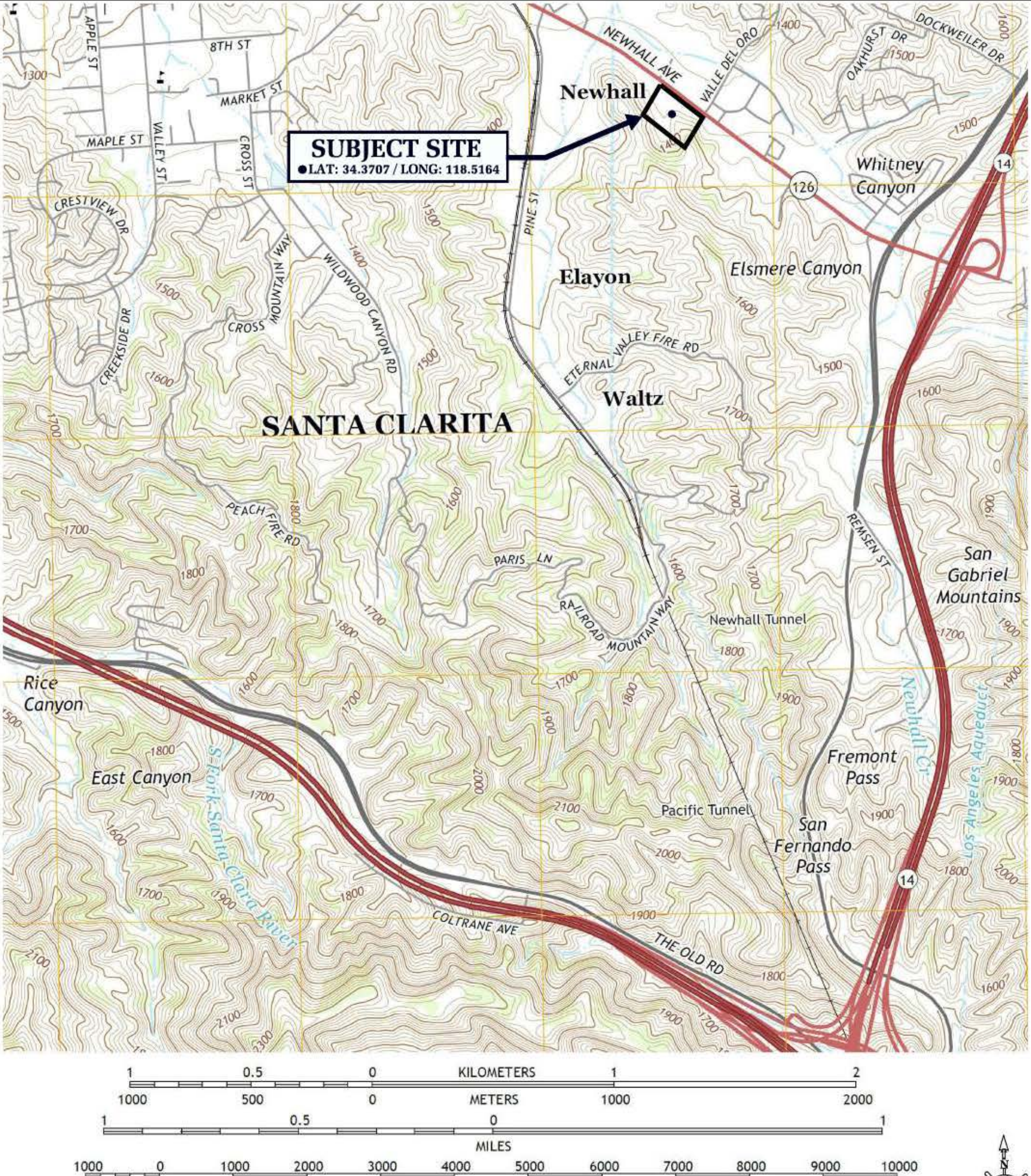

REFERENCE: U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, OAT MOUNTAIN, CA QUADRANGLE

VICINITY MAP

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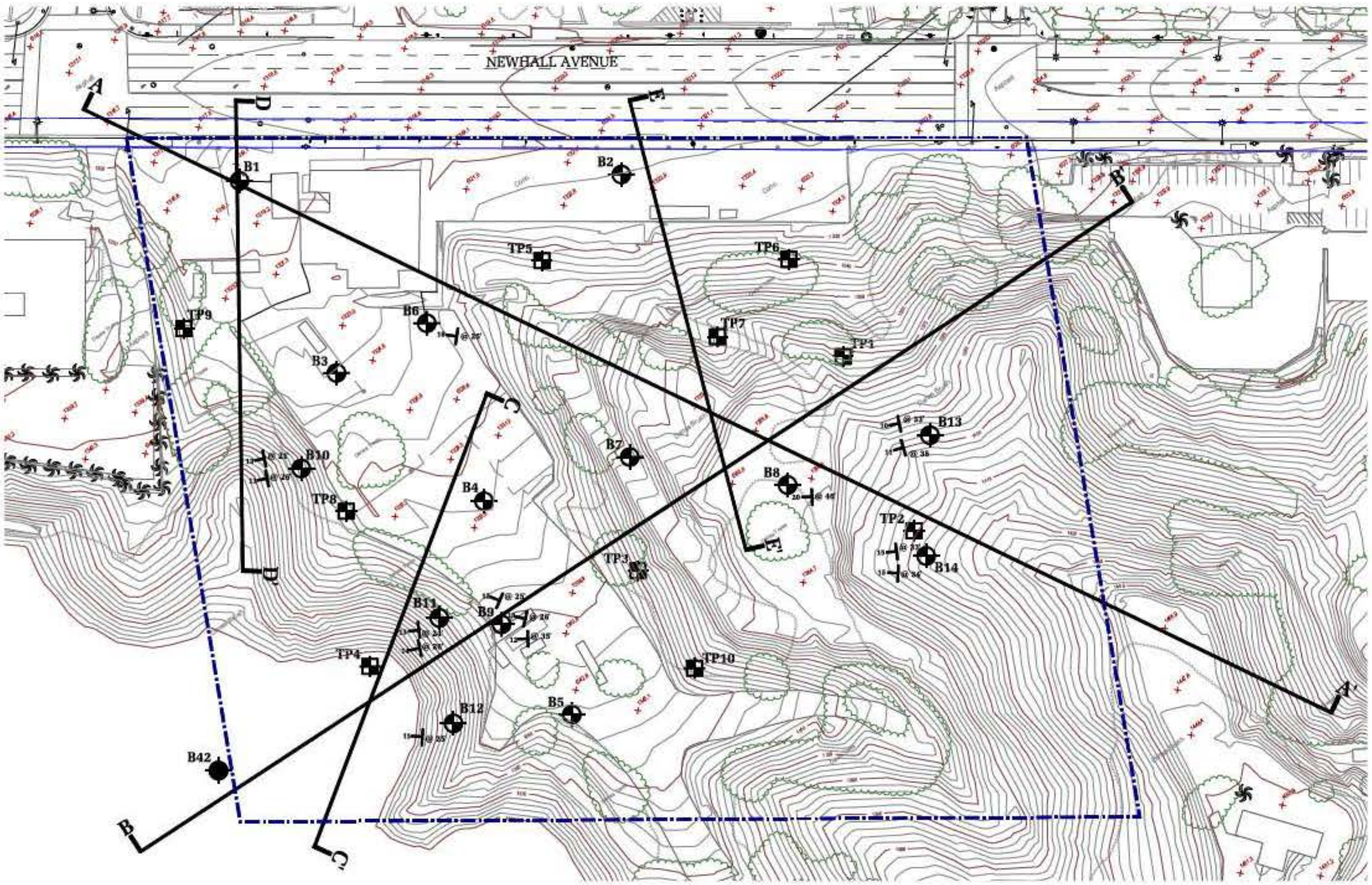
Consulting Geotechnical Engineers

CHANDLER PARTNERS

FILE NO. 22119



LEGEND

B14	LOCATION & NUMBER OF BORING BY GEOTECHNOLOGIES, INC.
TP10	LOCATION & NUMBER OF TEST PIT BY GEOTECHNOLOGIES, INC.
B42	LOCATION & NUMBER OF BORING BY R.T. FRANKIAN
E E'	CROSS SECTION
	SITE LIMITS
	CONTOUR OF EXISTING ELEVATION (2 FT INTERVALS)
af	FILL / TOP SOILS
Qa	ALLUVIUM
Qoa	OLDER ALLUVIUM
QTs	BEDROCK OF THE SAUGUS FORMATION
18	BEDROCK BEDDING STRIKE AND DIP

BORING B1	BORING B2	BORING B3	BORING B4	BORING B5	BORING B6	BORING B7
G.S. = 1319.1'	G.S. = 1322.0'	G.S. = 1324.2'	G.S. = 1332.0'	G.S. = 1343.0'	G.S. = 1327.0'	G.S. = 1361.0'
0' - 3' af	0' - 3' af	0' - 3' af	0' - 3' af	0' - 3' af	0' - 5' af	0' - 2' af
3' - 27.5' Qa	3' - 15' Qa	3' - 15' Qa	3' - 15' Qa	3' - 5' Qa	5' - 12' Qa	2' - 17' Qoa
27.5' - 50' Qoa	15' - 20' Qoa	15' - 20' Qoa	15' - 20' Qoa	53' - 25' Qoa	12' - 23' Qoa	17' - 30' QTs
					23' - 30' QTs	
					18 @ 25'	

BORING B8	BORING B9	BORING B10	BORING B11	BORING B12	BORING B13	BORING B14
G.S. = 1385.5'	G.S. = 1340.0'	G.S. = 1340.0'	G.S. = 1350.0'	G.S. = 1374.0'	G.S. = 1398.0'	G.S. = 1413.0'
0' - 3' af	0' - 3' af	0' - 2' af	0' - 2' af	0' - 3' af	0' - 1' af	0' - 1' af
3' - 35' Qoa	3' - 5' Qa	2' - 15' Qoa	2' - 22' Qoa	3' - 25' Qoa	1' - 31' Qoa	1' - 28' Qoa
35' - 50' QTs	5' - 25' Qoa	15' - 30' QTs	22' - 30' QTs	25' - 30' QTs	31' - 40' QTs	28' - 40' QTs
20 @ 46'	25' - 30' QTs	13 @ 21'	13 @ 24'	18 @ 25'	10 @ 33'	15 @ 33'
	12 @ 25'	13 @ 26'	14 @ 28'		11 @ 35	15 @ 34'
	13 @ 28'					
	12 @ 35'					

TEST PIT 1	TEST PIT 2	TEST PIT 3	TEST PIT 4	TEST PIT 5	TEST PIT 6	TEST PIT 7	TEST PIT 8	TEST PIT 9	TEST PIT 10
G.S. = 1371.0'	G.S. = 1409.0'	G.S. = 1351.0'	G.S. = 1386.0'	G.S. = 1334.0'	G.S. = 1337.0'	G.S. = 1359.0'	G.S. = 1332.0'	G.S. = 1328.0'	G.S. = 1355.0'
0' - 1' af	0' - 2' af	0' - 2' af	0' - 2' af	0' - 5' af	0' - 2' af	0' - 2.5' af	0' - 5' af	0' - 2' af	0' - 2' af
1' - 10' Qoa	2' - 10' Qoa	2' - 10' Qoa	2' - 10' Qoa	5' - 14' Qoa	2' - 14' Qoa	2.5' - 13'Qoa	5' - 10' Qoa	2' - 10' Qoa	2' - 10' Qoa

REFERENCE: TOOPGRAPHIC SURVEY PROVIDED BY ALLIANCE LAND PLANNING AND ENGINEERING, INC. NOT DATED

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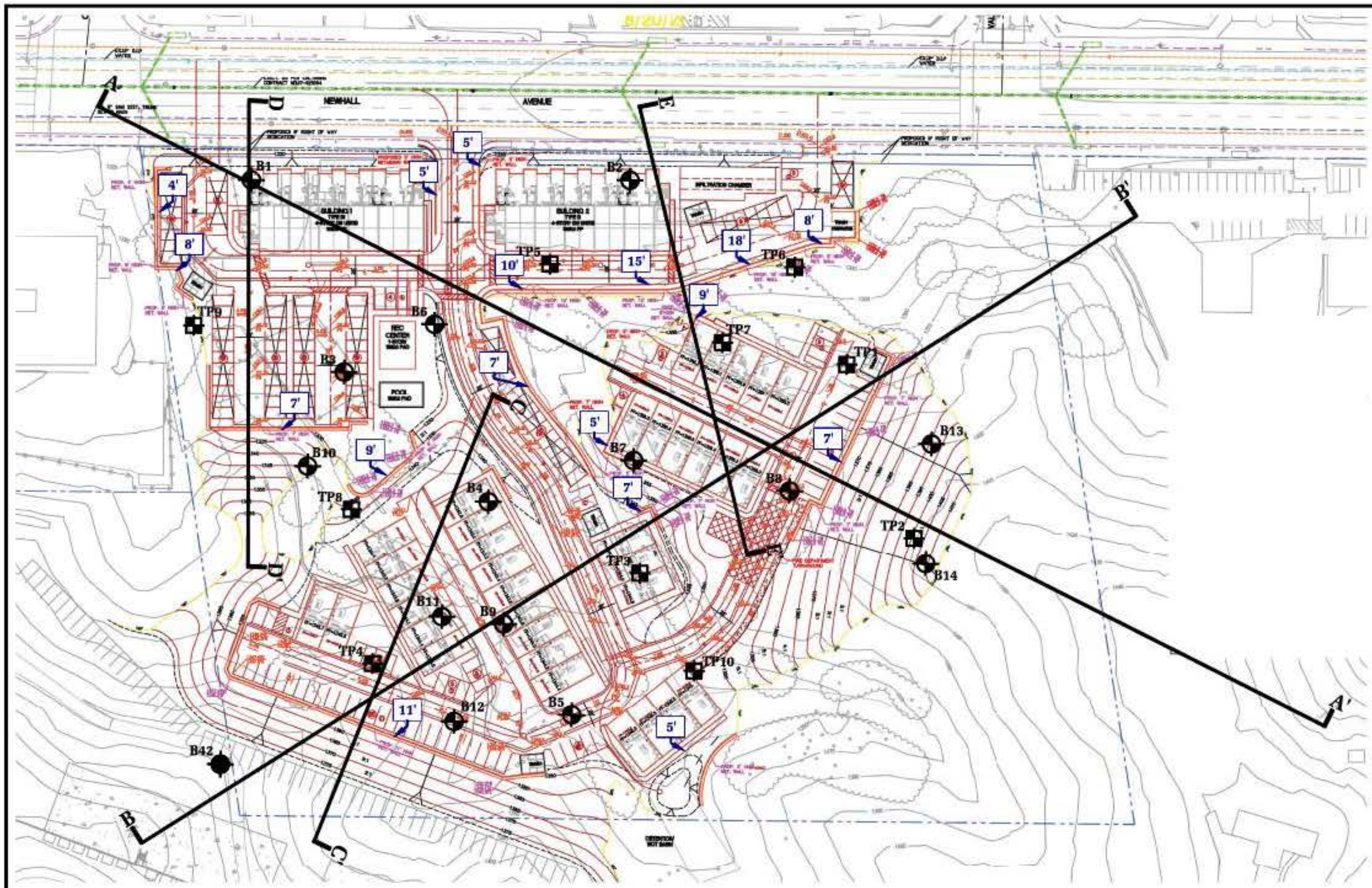
Consulting Geotechnical Engineers

GEOLOGIC MAP

CHANDLER PARTNERS

File No.: 22119

Date: February 2022



LEGEND

B14	LOCATION & NUMBER OF BORING BY GEOTECHNOLOGIES, INC.
TP10	LOCATION & NUMBER OF TEST PIT BY GEOTECHNOLOGIES, INC.
B42	LOCATION & NUMBER OF BORING BY R.T. FRANKIAN
E — E'	CROSS SECTION
— · —	SITE LIMITS
18'	HEIGHT OF PROPOSED RETAINING WALL (IN FEET)

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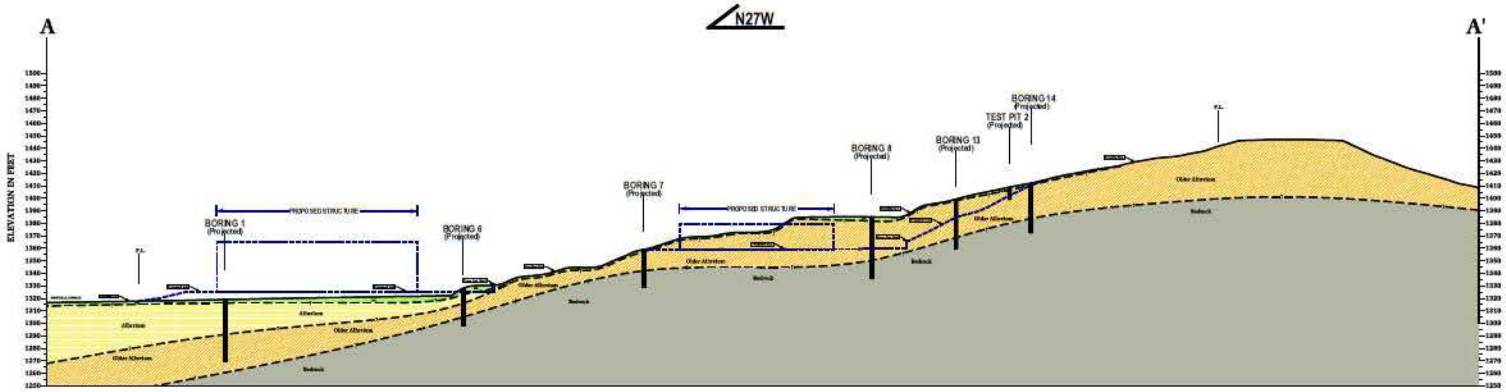
PROPOSED SITE PLAN

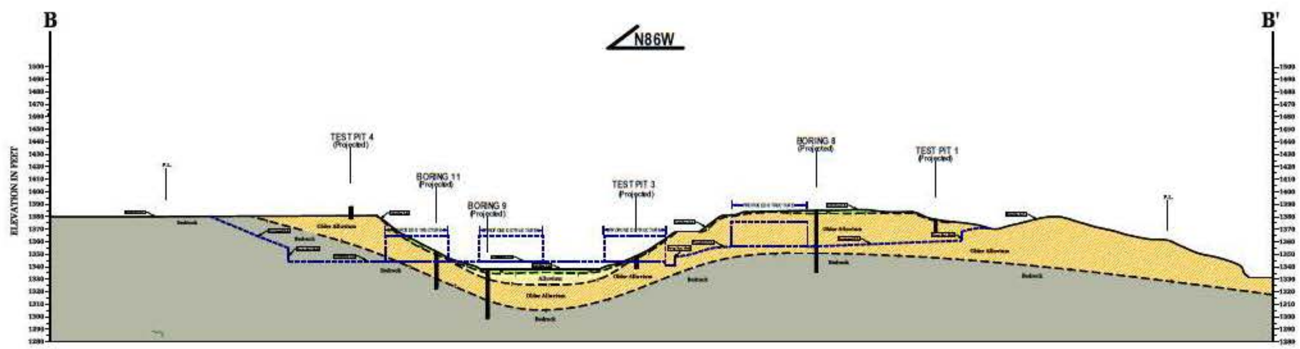
CHANDLER PARTNERS

File No.: 22119

Date: February 2022

REFERENCE: SITE DEVELOPMENT PLAN BY ALLIANCE LAND PLANNING AND ENGINEERING, INC. DATED AUGUST 20, 2021





REFERENCE: TOOPGRAPHIC SURVEY PROVIDED BY ALLIANCE LAND PLANNING AND ENGINEERING, INC. NOT DATED

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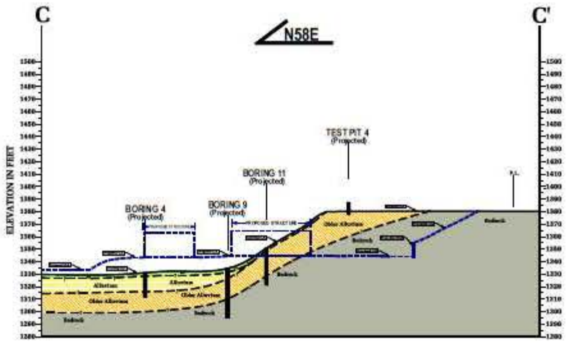
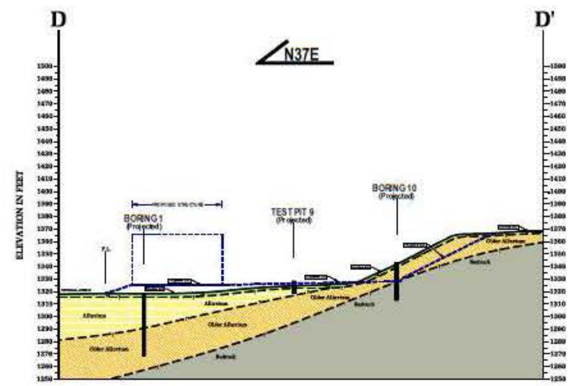
CROSS SECTIONS A-A' AND B-B'

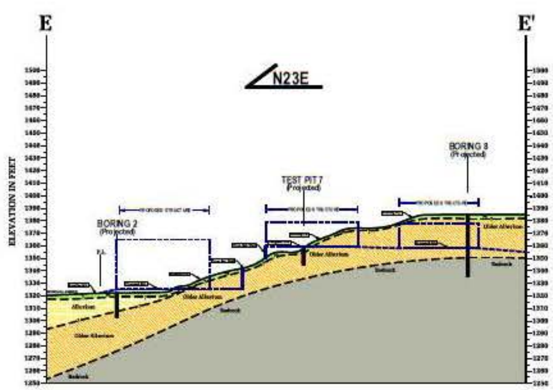
CHANDLER PARTNERS

File No.: 22119

Date: February 2022







SCALE IN FEET
0 30 50 100

REFERENCE: TOOPGRAPHIC SURVEY PROVIDED BY ALLIANCE LAND PLANNING AND ENGINEERING, INC. NOT DATED

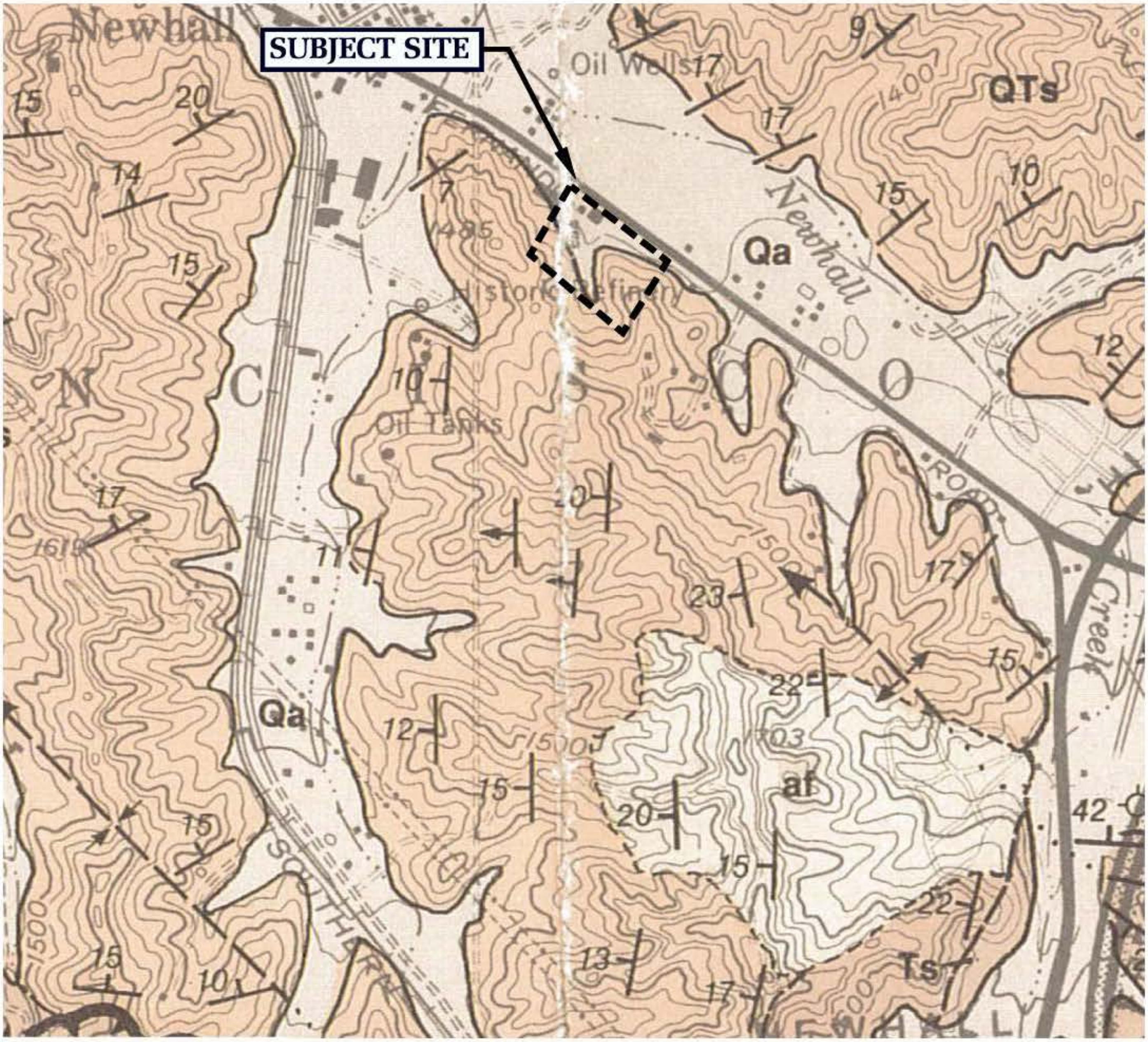
Geotechnologies, Inc.
Consulting Geotechnical Engineers

CROSS SECTIONS C-C', D-D' AND E-E'

CHANDLER PARTNERS

File No.: 22119

Date: February 2022



SCALE IN FEET

0 1000

LEGEND

af: Surficial Sediments - Artificial Cut and Fill

Qa: Surficial Sediments - Alluvial Gravel Sand and Clay of Valley and Flood Plain Areas

QTs: Bedrock of the Saugus Formation. Pebble-Cobble Conglomerate, Sandstone, and lesser amount of Siltstone-Claystone

N

REFERENCE: DIBBLEE, T.W., (1976) GEOLOGIC MAP OF THE OAT MOUNTAIN AND CANOGA PARK QUADRANGLE (#DF-236

LOCAL GEOLOGIC MAP - DIBBLEE

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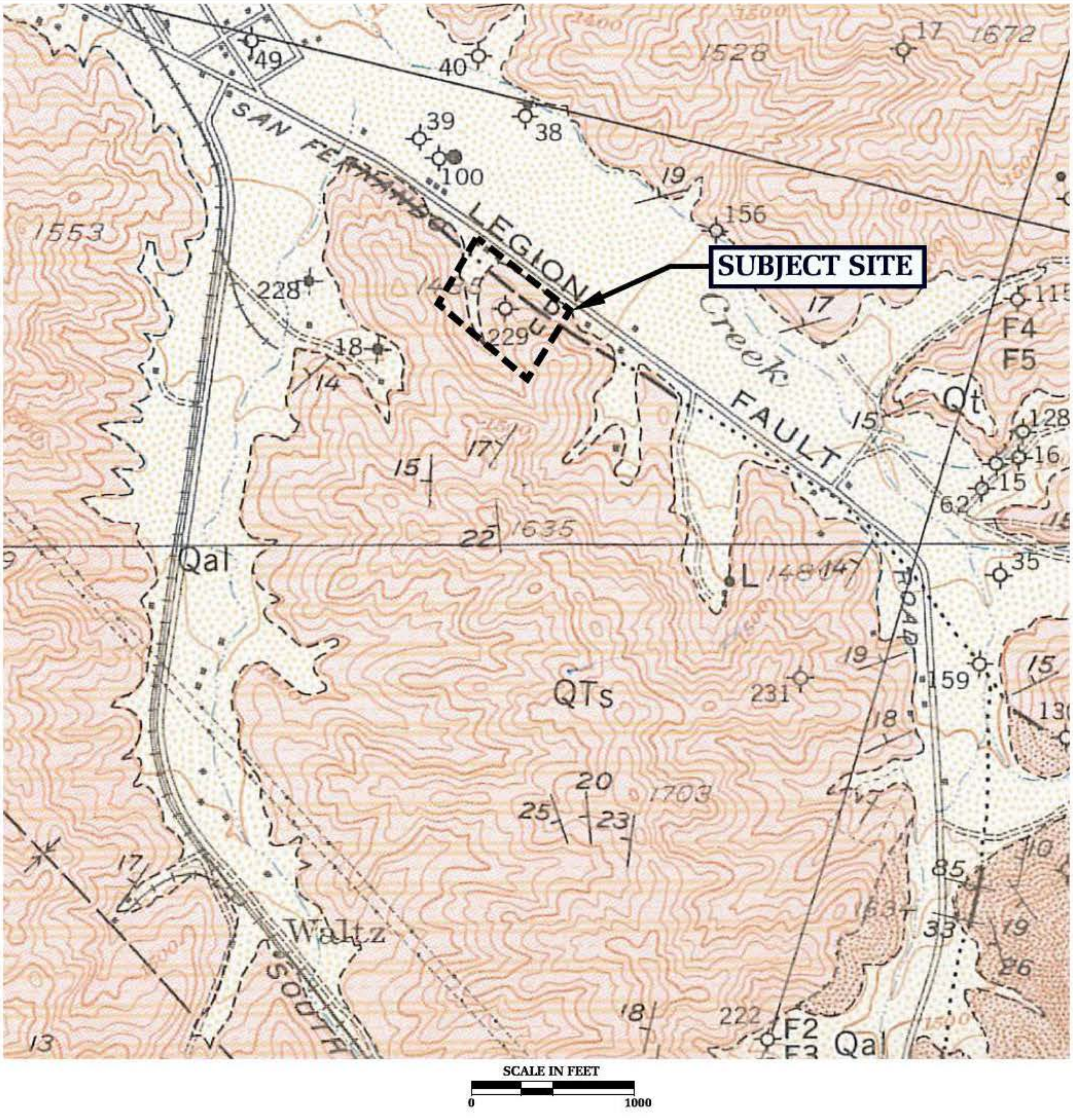

LEGEND

Qal: Alluvium: Clay, sand, and gravel, unconsolidated; gray, light-brown, or reddish-brown

QTs: Saugus Formation: Sandstone and conglomerate, brown and reddish brown

—••••? Fault - dashed where indefinite or inferred, dotted where concealed, queried where existence is doubtful

● Producing Oil Well

○ Well Bing Drilled

REFERENCE: GEOLOGY OF VENTURA BASIN, LOS ANGELES COUNTY, CA, WINTERER & DURHAM, 1962, USGS PP334-H

LOCAL GEOLOGIC MAP - WINTERER & DURHAM

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Consulting Geotechnical Engineers

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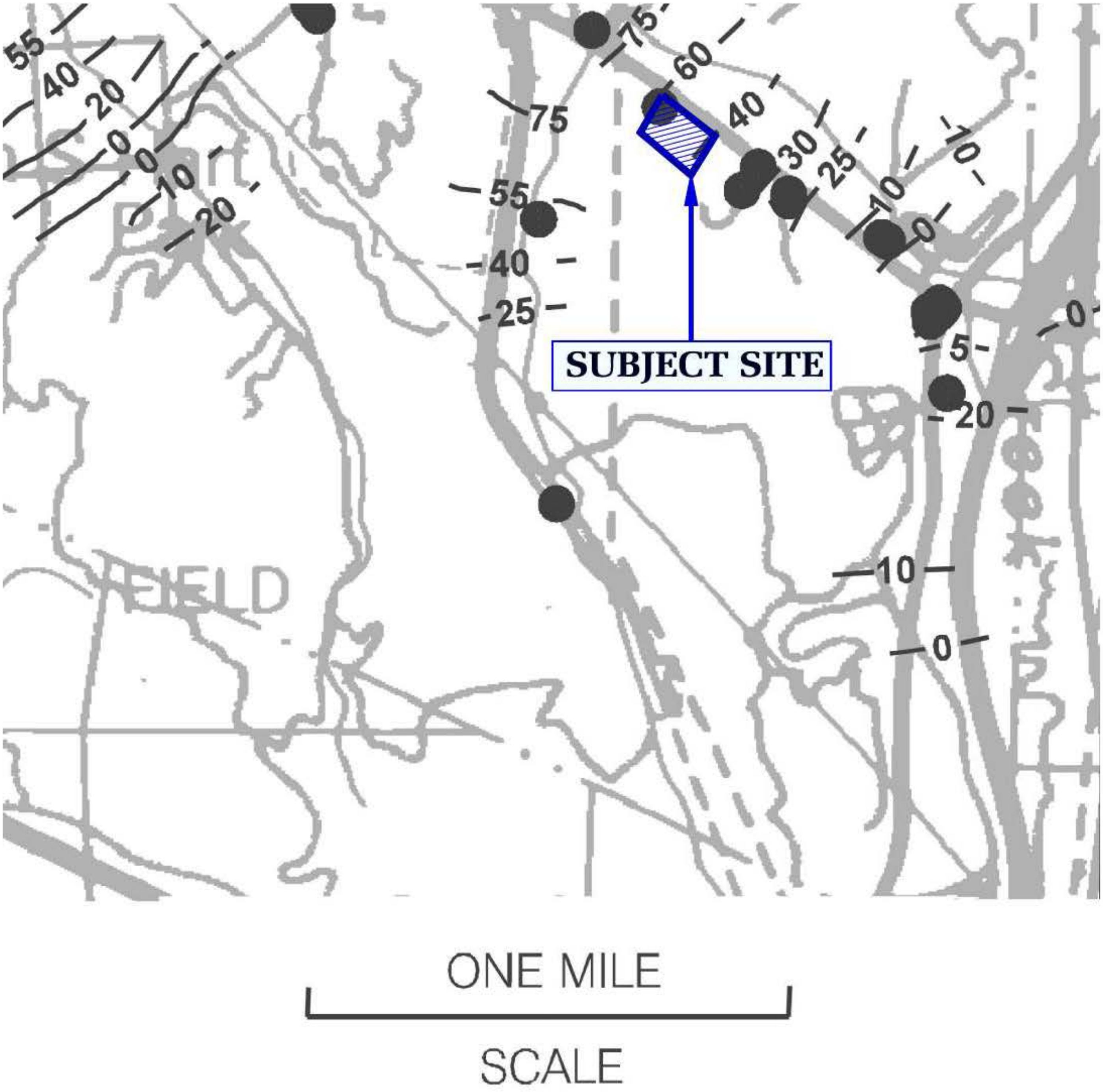

20 Depth to groundwater in feet

REFERENCE: CDMG, SEISMIC HAZARD ZONE REPORT, 05
OAT MOUNTAIN 7.5 - MINUTE QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA (1997, REVISED 2006)

HISTORICALLY HIGHEST GROUNDWATER LEVELS

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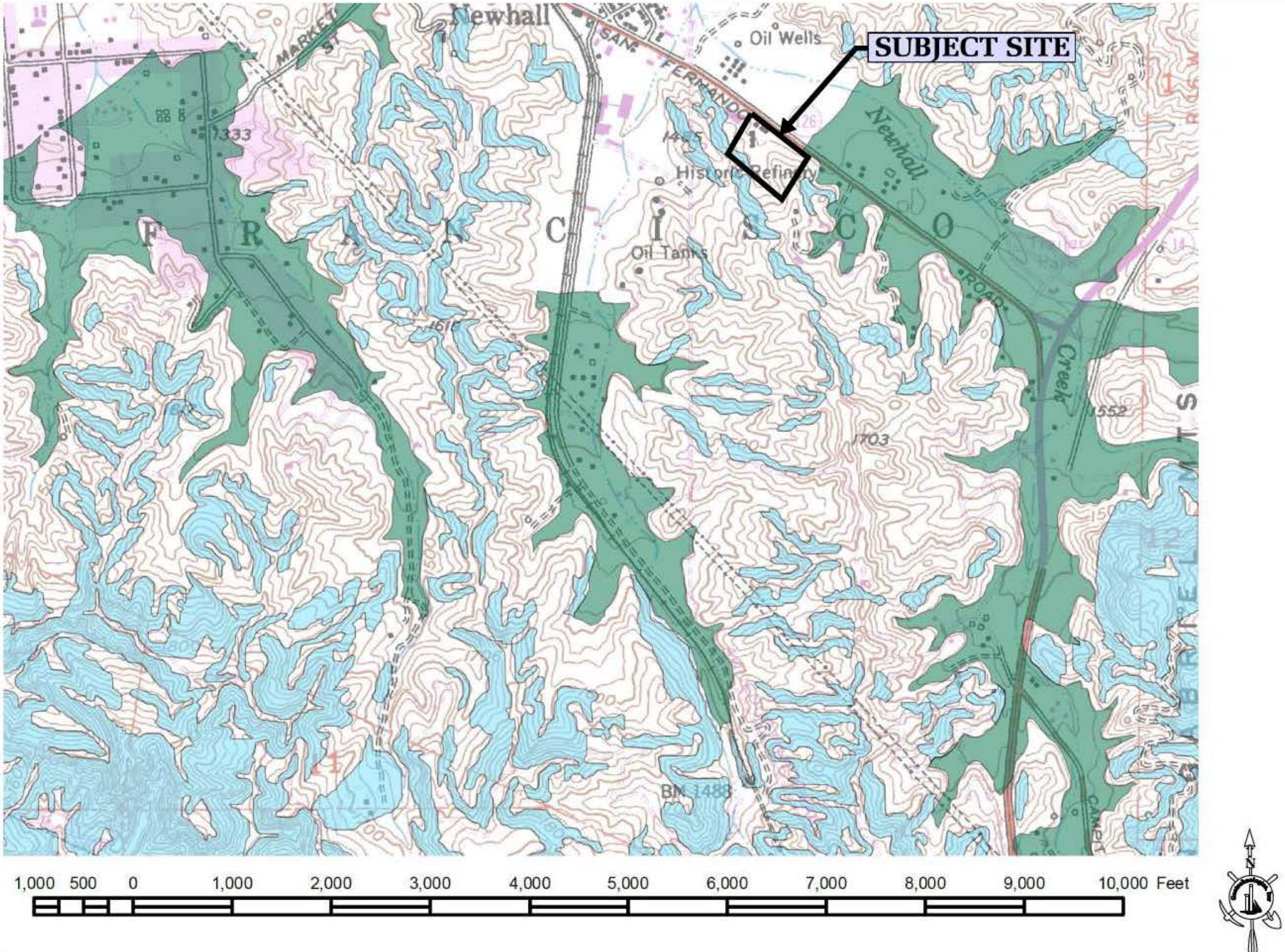
FILE NO. 22119



Earthquake Fault Zones
Alquist-Priolo Earthquake Fault Zone

Liquefaction Zones

Earthquake-Induced Landslide Zones

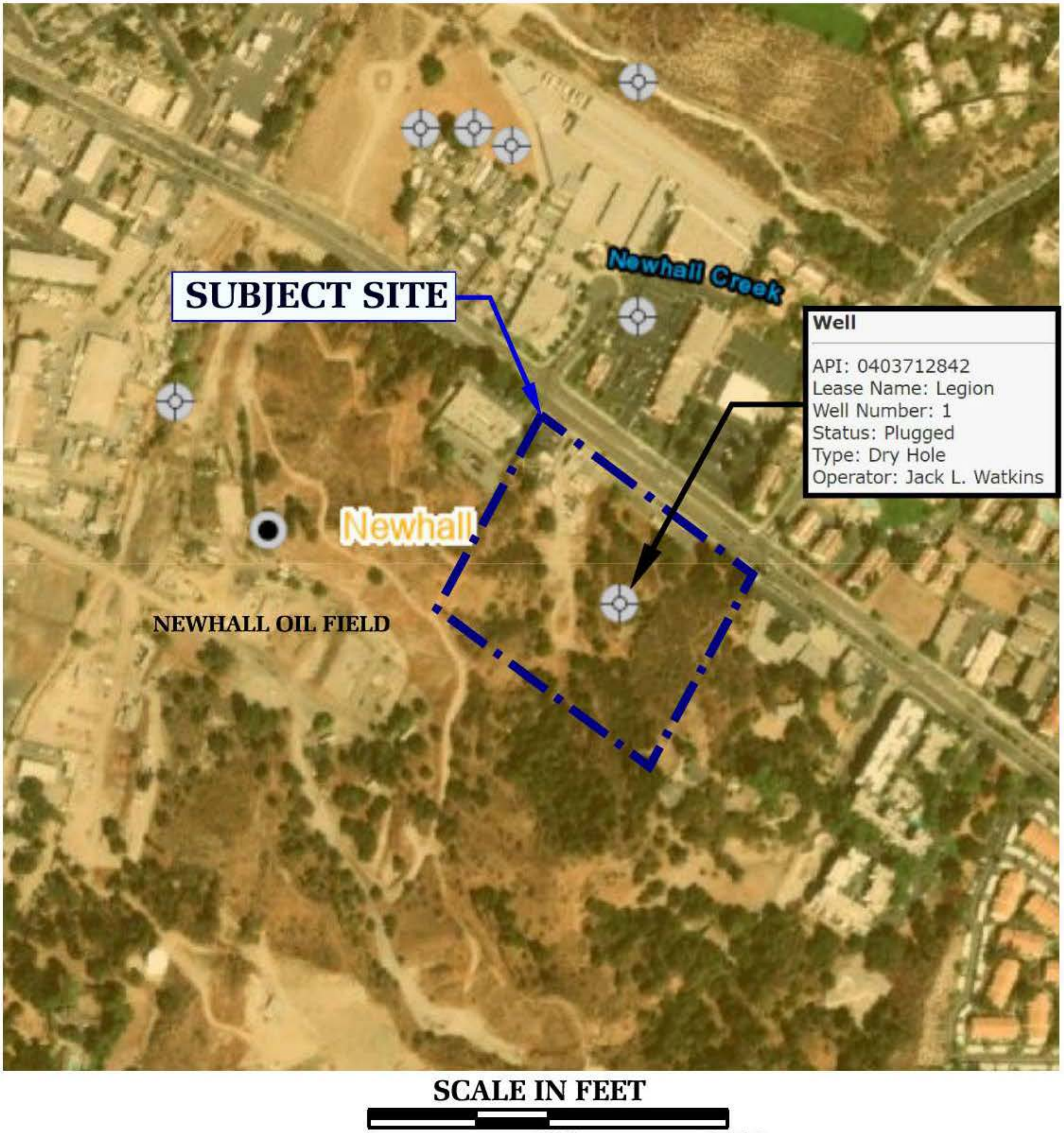
REFERENCE: EARTHQUAKE ZONES OF REQUIRED INVESTIGATION, OAT MOUNTAIN QUADRANGLE, CALIFORNIA GEOLOGICAL SURVEY, 1976 AND 1998

EARTHQUAKE ZONES OF REQUIRED INVESTIGATION MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

CHANDLER PARTNERS

FILE NO. 22119



REFERENCE: https://maps.conservation.ca.gov/doggr/wellfinder/#openModal

OIL FIELD AND OIL WELL MAP

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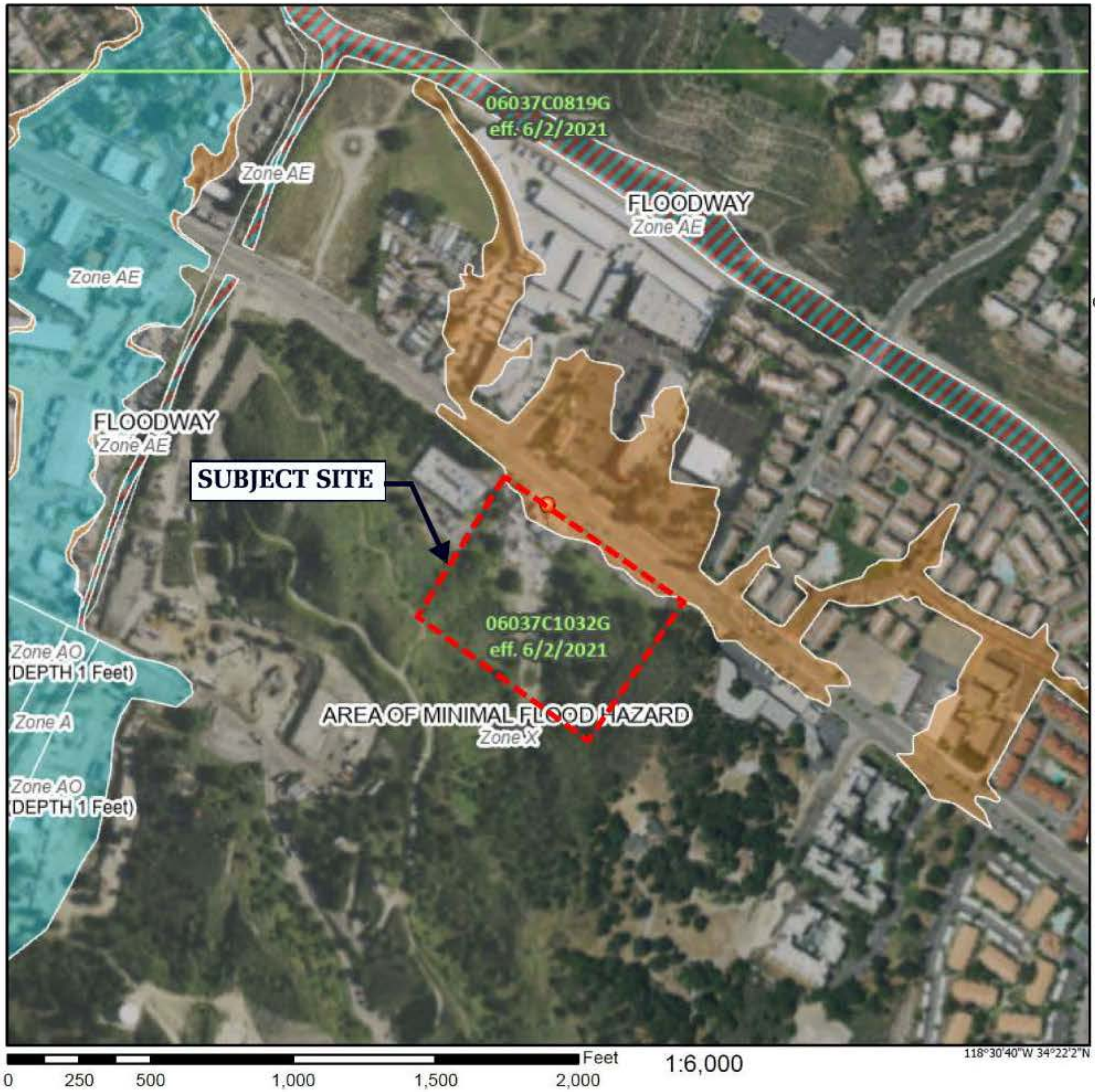
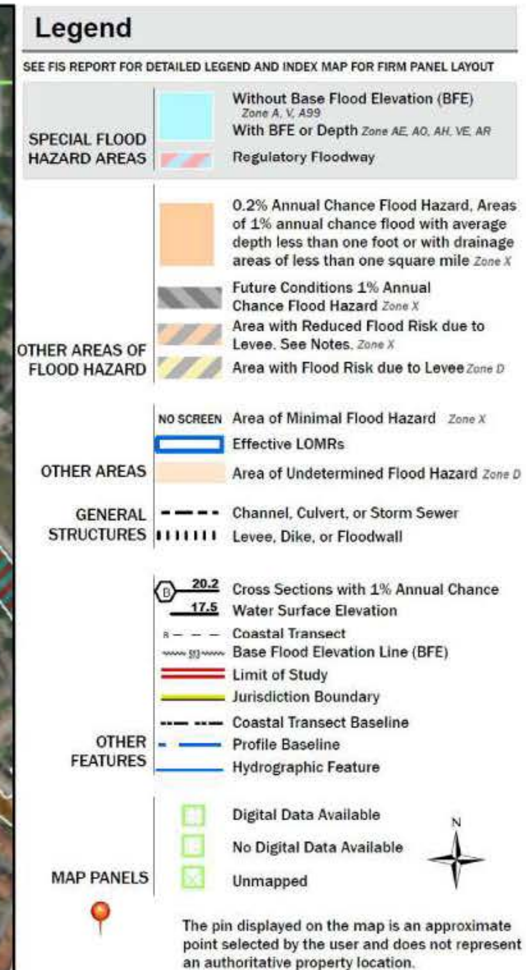
CHANDLER PARTNERS

FILE NO. 22119





FLOOD INSURANCE RATE MAP	
Geotechnologies, Inc. Consulting Geotechnical Engineers	CHANDLER PARTNERS
	FILE No. 22119

BORING LOG NUMBER 1

Chandler Partners **Date: 04/19/21** **Elevation: 1,319.1'***

File No. 22119 **Method: 8-inch diameter Hollow Stem Auger**

dy/km *Reference: Topographic Survey provided by Alliance, not dated

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0 --		4-inch Asphalt, No Base
				-		
				1 --		FILL: Silty Sand, dark brown, moist, medium dense, fine grained
				-		
				2 --		
				-		
2.5	23	8.8	121.4	3 --		
				-	SM	ALLUVIUM: Silty Sand, dark brown, moist, medium dense, fine grained
				4 --		
				-		
5	23	11.7	SPT	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	38	9.8	124.4	-		
				8 --		
				-		
				9 --		
				-		
10	30	8.8	SPT	10 --		
				-		
				11 --		
				-		
				12 --		
12.5	51	9.3	113.0	-		
				13 --	SM/SP	Silty Sand to Sand, dark brown, moist, medium dense, fine grained
				-		
				14 --		
				-		
15	34	10.6	SPT	15 --		
				-		
				16 --		
				-		
				17 --		
17.5	53	11.4	123.8	-		
				18 --	SM	Silty Sand, dark brown, moist, medium dense, fine grained
				-		
				19 --		
				-		
20	34	12.3	SPT	20 --		
				-	SM/SP	Silty Sand to Sand, dark brown, moist, medium dense, fine grained
				21 --		
				-		
				22 --		
22.5	100/8"	10.6	102.2	-		
				23 --	SM	Silty Sand with rock fragments, dark brown, moist, very dense, fine grained
				-		
				24 --		
				-		
25	41	12.7	SPT	25 --		
				-		

BORING LOG NUMBER 1

Chandler Partners

File No. 22119

dy/km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
27.5	58	6.8	122.4			
				28	SP	OLD ALLUVIUM: Sand, dark brown, moist, dense, fine to medium grained, minor cobbles
				29		
30	43	11.2	SPT	30		
					SM/SP	Silty Sand to Sand, dark brown, moist, dense, fine to medium grained
				31		
				32		
32.5	70	8.7	122.4			
				33	SM	Silty Sand, dark brown, moist, dense, fine grained, minor cobbles
				34		
35	42	10.7	SPT	35		
					SM/SP	Silty Sand to Sand, dark brown, moist, dense, fine grained
				36		
				37		
37.5	69	4.8	116.3			
				38	SM	Silty Sand, dark brown, moist, dense, fine grained
				39		
40	43	10.8	SPT	40		
					SM/SP	Silty Sand, dark brown, moist, medium dense, fine to medium grained
				41		
				42		
42.5	83	8.9	118.9			NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual. Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted SPT=Standard Penetration Test
				43		
				44		
45	74	8.0	SPT	45		
				46		
				47		
47.5	90	4.4	117.4			
				48	SP	Sand, dark and yellowish brown, moist, very dense, fine to medium grained, minor cobbles
				49		
50	29 50/5"	2.8	SPT	50		
						Total Depth 50 feet No Water Fill to 3 feet

BORING LOG NUMBER 2

Chandler Partners | **Date: 04/19/21** | **Elevation: 1,322.0'***

File No. 22119 | **Method: 8-inch diameter Hollow Stem Auger**

dy/km | *Reference: Topographic Survey provided by Alliance, not dated

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
				0		2-inch Asphalt, No Base
				1		FILL: Silty Sand, dark brown, moist, medium dense, fine grained
				2		
2.5	16	10.1	116.6			
				3	SM	ALLUVIUM: Silty Sand, dark brown, moist, medium dense, fine grained
				4		
5	19	7.2	114.4	5		
				6		
				7		
				8		
				9		
10	27	11.1	122.3	10		
				11		
				12		
				13		
				14		
15	49	11.2	113.6	15	SM/SW	OLD ALLUVIUM: Silty Sand to Sand, dark brown, slightly moist, dense, fine to coarse grained
				16		
				17		
				18		
				19		
20	40	7.2	104.8	20		Total Depth 20 feet No Water Fill to 3 feet
				21		
				22		
				23		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				24		
				25		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted

BORING LOG NUMBER 3

Chandler Partners | **Date: 04/19/21** | **Elevation: 1,324.2'***

File No. 22119 | **Method: 8-inch diameter Hollow Stem Auger**

dy/km | *Reference: Topographic Survey provided by Alliance, not dated

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
				0		FILL: Silty Sand, dark brown, moist, medium dense, fine grained
				1		
				2		
2.5	22	8.0	109.2			
				3	SM	ALLUVIUM: Silty Sand, dark brown, moist, medium dense, fine grained
				4		
5	32	7.1	111.1	5	SM/SW	Silty Sand to Sand, dark brown, moist, dense, fine to coarse grained
				6		
				7		
				8		
				9		
10	39	5.7	114.5	10		
				11		
				12		
				13		
				14		
15	72	8.7	115.8	15	SM/SP	OLD ALLUVIUM: Silty Sand to Sand, dark brown, moist, dense, fine grained
				16		
				17		
				18		
				19		few cobbles
20	76	10.0	121.5	20		Total Depth 20 feet No Water Fill to 3 feet
				21		
				22		
				23		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				24		
				25		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted

BORING LOG NUMBER 4

Chandler Partners — **Date: 04/19/21** — **Elevation: 1,332.0'***

File No. 22119 — **Method: 8-inch diameter Hollow Stem Auger**

dy/km — *Reference: Topographic Survey provided by Alliance, not dated

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
				0		FILL: Silty Sand, dark brown, moist, medium dense, fine grained
				1		
2.5	36	6.3	106.8	2		
				3	SM	ALLUVIUM: Silty Sand, dark brown, moist, medium dense, fine grained
				4		
5	44	7.1	111.9	5		
				6		
				7		
				8		
				9		
10	56	5.0	119.0	10		
				11		
				12		
				13		
				14		
15	49 50/5"	8.3	121.2	15	SM/SW	OLD ALLUVIUM: Silty Sand to Sand, dark brown, moist, very dense, fine to coarse grained, with some cobbles
				16		
				17		
				18		
				19		
20	100/7"	2.9	114.4	20		Total Depth 20 feet No Water Fill to 3 feet
				21		
				22		
				23		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				24		
				25		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted

BORING LOG NUMBER 5

Chandler Partners — **Date: 04/19/21** — **Elevation: 1,345.0'***

File No. 22119 — **Method: 8-inch diameter Hollow Stem Auger**

dy/km — *Reference: Topographic Survey provided by Alliance, not dated

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
				0		FILL: Silty Sand, dark brown, moist, medium dense, fine grained
				1		
				2		
2.5	30 50/5"	5.6	122.9			
				3	SM	ALLUVIUM: Silty Sand with rock fragments, dark brown, moist, very dense, fine grained
				4		
5	100/8"	4.8	119.6	5	SP/SW	OLD ALLUVIUM: Sand, yellowish brown, moist, very dense, fine to medium grained, minor cobbles, rock fragments
				6		
				7		
				8		
				9		
10	100/8"	5.5	100.0	10		
				11		
				12		
				13		
				14		
15	100/8"	5.1	106.7	15		
				16		
				17		
				18		
				19		
20	100/8"	4.8	101.2	20		
				21		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				22		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				23		
				24		
25	100/8"	8.6	96.7	25		
						Total Depth 25 feet; No Water; Fill to 3 feet

BORING LOG NUMBER 6

Chandler Partners | **Date: 04/30/21** | **Elevation: 1,327.0'***

File No. 22119 | **Method: 24-inch Bucket Auger**

dy/kn | *Reference: Topographic Survey provided by Alliance, not dated

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
				0		FILL: Silty Sand, dark brown, moist, medium dense, fine grained, few cobbles
				1		
				2		
				3		
				4		
5	Push/ 6" 1/6"	9.6	97.3	5	SM	ALLUVIUM: Silty Sand, dark brown, moist, medium dense, fine grained, few cobbles
				6		
				7		
				8		
				9		
10	1/6" 1/6"	9.7	113.3	10		
				11		
				12	SM/SP	OLD ALLUVIUM: Silty Sand to Sand, dark and yellowish brown, slightly moist, dense, fine grained, few cobles
				13		
				14		
15	2/6" 2/6"	9.7	116.0	15		
				16		
				17		
				18		
				19		
20	1/6" 1/6"	9.1	120.2	20		
				21		
21.5	1/6" 3/6"	No Recovery		22		
				23		BEDROCK (SAUGUS FORMATION): Sandstone, yellow and grayish brown, slightly moist, hard, fine to coarse grained
				24		
25	5/6" 12/6"	6.2	124.5	25		@25' Bedding [N45E, 18NW]

BORING LOG NUMBER 6

Chandler Partners

File No. 22119
dy/kn

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		Sandstone and conglomerate
				27		
27.5	41	No Recovery				
				28		
				29		Sandstone, light gray, moist, hard
30	5/6" 7/5"	6.7	116.5	30		
						Total Depth 30 feet No Water Fill to 5 feet
				31		
				32		
				33		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				34		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				35		
				36		Kelly Bar Weights: 0 - 26': 3,390 lb. 26' - 52': 2,230 lb.
				37		
				38		
				39		
				40		
				41		
				42		
				43		
				44		
				45		
				46		
				47		
				48		
				49		
				50		

BORING LOG NUMBER 7

Chandler Partners **Date: 04/29/21** **Elevation: 1,361.0'***

File No. 22119 **Method: 24-inch Bucket Auger**

dy/km *Reference: Topographic Survey provided by Alliance, not dated

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
				0		FILL: Silty Sand to Sand, dark brown, moist, medium dense, fine grained
				1		
				2		
				3	SM/SW	OLD ALLUVIUM: Silty Sand to Sand, yellowish brown to gray, moist, dense, fine to coarse grained
				4		
5	3/6" 6/6"	7.2	126.3	5		
				6		
				7		
				8		
				9		
10	3/6" 4/6"	7.2	115.3	10		
				11		
				12		
				13		
				14		
15	2/6" 6/6"	8.2	117.1	15		
				16		
				17		
				18		BEDROCK (SAUGUS FORMATION): Sandstone, yellowish to grayish brown, slightly moist, moderately hard, fine to coarse grained
				19		
20	2/6" 3/6"	10.0	129.0	20		
				21		
				22		
				23		
				24		
25	2/6" 4/6"	11.2	121.7	25		Sandstone and conglomerate

BORING LOG NUMBER 7

Chandler Partners

File No. 22119

dy/kn

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
				28		
				29		
30	3/6" 6/6"	8.5	114.4	30		
				31		Total Depth 30 feet No Water Fill to 2 feet
				32		
				33		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				34		
				35		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				36		
				37		Kelly Bar Weights: 0 - 26': 3,390 lb. 26' - 52': 2,230 lb.
				38		
				39		
				40		
				41		
				42		
				43		
				44		
				45		
				46		
				47		
				48		
				49		
				50		

BORING LOG NUMBER 8

Chandler Partners | **Date: 04/29/21** | **Elevation: 1,385.5'***

File No. 22119 | **Method: 24-inch Bucket Auger**

dy/km

*Reference: Topographic Survey provided by Alliance, not dated

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
				0		FILL: Silty Sand to Sand, dark brown, moist, medium dense, fine grained
				1		
				2		
				3		
				4	SM/SP	OLD ALLUVIUM: Silty Sand to Sand, dark brown, moist, dense, fine to coarse grained with few cobbles
5	4/6" 5/6"	6.3	114.9	5		
				6	SM/SW	Silty Sand to Sand, dark and yellowish brown, moist, dense, fine to coarse grained, with pebbles
				7		
				8		
				9		
10	3/6" 5/6"	4.6	109.5	10		dark brown and light gray, with cobbles
				11		
				12		
				13		
				14		
15	3/6" 5/6"	5.0	121.1	15		yellowish brown
				16		
				17		
				18		
				19		
20	2/6" 4/6"	5.9	104.0	20		
				21		
				22		
				23		
				24		
25	2/6" 3/6"	7.7	122.7	25		very dense

BORING LOG NUMBER 8

Chandler Partners

File No. 22119

dy/km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
				28		
				29		
30	3/6" 6/6"	8.9	122.1	30		
				31		
				32		
				33		
				34		
35	10/6" 11/5"	10.3	131.5	35		
				36		BEDROCK (SAUGUS FORMATION): Sandstone, dark and yellowish brown, slightly moist, moderately hard, fine to coarse grained
				37		
				38		
				39		
40	6/6" 13/6"	5.5	122.7	40		
				41		Sandstone, dark brown, moist, hard
				42		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				43		
				44		Kelly Bar Weights: 0 - 26': 3,390 lb. 26' - 52': 2,230 lb.
45	10/6" 11/6"	7.7	122.6	45		
				46		@46' Bedding [N30E, 20NW]
				47		
				48		
				49		
50	26/7"	6.6	109.5	50		conglomerate, gray, moist, hard
						Total Depth 50 feet No Water Fill to 35 feet

BORING LOG NUMBER 9

Chandler Partners | **Date: 04/30/21** | **Elevation: 1,340.0'***

File No. 22119 | **Method: 24-inch Bucket Auger**

dy/km

*Reference: Topographic Survey provided by Alliance, not dated

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
				0 --		FILL: Silty Sand to Sand, dark brown, moist, medium dense, fine grained
				1 --		
				2 --		
				3 --	SM	ALLUVIUM: Silty Sand, dark brown, moist, dense, fine grained, few cobbles
				4 --		
5	3/6" 4/6"	6.8	127.4	5 --	SW	OLD ALLUVIUM: Sand, yellowish brown, moist, dense, fine to coarse grained, few cobbles
				6 --		
				7 --		
				8 --		
				9 --		
10	3/6" 6/6"	4.3	121.6	10 --		
				11 --	SM/SW	Silty Sand to Sand, light gray, moist, dense, fine to coarse grained
				12 --		
				13 --		
				14 --		
15	3/6" 9/6"	5.3	118.3	15 --		
				16 --		
				17 --		
				18 --		
				19 --		
20	5/6" 7/6"	8.6	109.6	20 --		yellowish brown
				21 --		
				22 --		
				23 --		
				24 --		BEDROCK (SAUGUS FORMATION): Sandstone and conglomerate, grayish brown, slightly moist, moderately hard, fine to coarse grained
25	7/6" 5/6"	7.8	125.6	25 --		@25' Bedding [N60E, 12NW]

BORING LOG NUMBER 9

Chandler Partners

File No. 22119

dy/km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		@26' Bedding [N50E, 13NW]
				27		
				28		
				29		
30	9/6" 9/3"	3.7	128.7	30		
				31		
				32		
				33		
				34		
35	7/6" 13/3"	6.6	124.5	35		@35' Bedding [N35E, 12NW]
				36		
				37		
				38		
				39		
40	9/6" 10/1"	5.8	125.1	40		Total Depth 40 feet No Water Fill to 3 feet
				41		
				42		
				43		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				44		
				45		Kelly Bar Weights: 0 - 26': 3,390 lb. 26' - 52': 2,230 lb.
				46		
				47		
				48		
				49		
				50		

BORING LOG NUMBER 10

Chandler Partners **Date: 09/30/21** **Elevation: 1,340.0'***

File No. 22119 **Method: 24-inch Fly Auger**

km

*Reference: Topographic Survey provided by Alliance, not dated

	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
	0 --		TOP SOIL: Silty Sand, dark brown, slightly moist, medium dense, fine grained
	1 --		
	2 --	SM	OLD ALLUVIUM: Silty Sand, yellowish brown, slightly moist, dense, fine to coarse grained, with abundant cobble
	3 --		
	4 --		
	5 --		moist, dense
	6 --		
	7 --		
	8 --		
	9 --		
	10 --		
	11 --		
	12 --		
	13 --		
	14 --		
	15 --		BEDROCK (SAUGUS FORMATION): Sandy Siltstone, yellow to grayish brown, slightly moist, moderately hard, fine to coarse grained, with cobbles
	16 --		
	17 --		
	18 --		
	19 --		
	20 --		
	21 --		@21' Bedding [N25E, 13NW]
	22 --		
	23 --		
	24 --		
	25 --		

BORING LOG NUMBER 10

Chandler Partners

File No. 22119

km

	Depth in feet	USCS Class.	Description
	26		@26' Bedding [N25E, 13NW]
	27		
	28		
	29		
	30		
	31		Total Depth 30 feet No Water Fill to 2 feet
	32		
	33		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
	34		
	35		
	36		
	37		
	38		
	39		
	40		
	41		
	42		
	43		
	44		
	45		
	46		
	47		
	48		
	49		
	50		

BORING LOG NUMBER 11

Chandler Partners | **Date: 09/30/21** | **Elevation: 1,350.0'***

File No. 22119 | **Method: 24-inch Fly Auger**

km

*Reference: Topographic Survey provided by Alliance, not dated

	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
	0		TOP SOIL: Silty Sand, dark brown, moist, medium dense, fine grained, with few cobbles
	1		
	2		
		SM/SW	OLD ALLUVIUM: Silty Sand to Sand, dark and yellowish brown, slightly moist, dense, fine to coarse grained, with cobbles
	3		
	4		
	5		
	6		
	7		
	8		dark brown, few cobbles
	9		
	10		
			dark grayish brown, dense
	11		
	12		
			gray to grayish brown
	13		
	14		
	15		
	16		
	17		
	18		
	19		
	20		
	21		
			abundant gravel
	22		
			BEDROCK (SAUGUS FORMATION): Sandstone, gray to grayish brown, slightly moist, moderately hard, fine to coarse grained @22' Bedding [N25E, 15NW]
	23		
	24		@24' Bedding [N30E, 13NW]
	25		

BORING LOG NUMBER 11

Chandler Partners

File No. 22119

km

	Depth in feet	USCS Class.	Description
	26		
	27		Sandstone with pebbles, yellow and grayish brown @26' Bedding [N25E, 14NW]
	28		
	29		
	30		
	31		Total Depth 30 feet No Water No Fill
	32		
	33		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
	34		
	35		
	36		
	37		
	38		
	39		
	40		
	41		
	42		
	43		
	44		
	45		
	46		
	47		
	48		
	49		
	50		

BORING LOG NUMBER 12

Chandler Partners | **Date: 09/30/21** | **Elevation: 1,374.0'***

File No. 22119 | **Method: 24-inch Fly Auger**

km

*Reference: Topographic Survey provided by Alliance, not dated

	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
	0		TOP SOIL: Silty Sand, dark brown, moist, medium dense, fine grained
	1		
	2		
	3		
	4	SM/SW	OLD ALLUVIUM: Silty Sand to Sand, yellowish brown, slightly moist, dense, fine to coarse grained
	5		
	6		
	7		
	8		
	9		grayish brown, dense, few cobbles and pebbles
	10		
	11		
	12		
	13		
	14		
	15		
	16		
	17		
	18		
	19		
	20		
	21		
	22		
	23		
	24		
	25		BEDROCK (SAUGUS FORMATION): Sandstone, grayish brown, slightly moist, moderately hard, fine to coarse grained, with pebbles and cobbles

BORING LOG NUMBER 12

Chandler Partners

File No. 22119
km

	Depth in feet	USCS Class.	Description
	26 27 28 29		@25' Bedding [N40E, 15NW]
	30 31 32 33 34 35 36 37 38 39 40 41 42 43 44 45 46 47 48 49 50		Total Depth 30 feet No Water No Fill NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.

BORING LOG NUMBER 13

Chandler Partners | **Date: 10/01/21** | **Elevation: 1,398.0'***

File No. 22119 | **Method: 24-inch Fly Auger**

km

*Reference: Topographic Survey provided by Alliance, not dated

	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
	0 --		TOP SOIL: Silty Sand, dark brown, moist, fine to medium grained, few cobbles
	1 --	SM/SW	OLD ALLUVIUM: Silty Sand to Sand, yellow to grayish brown, slightly moist, dense, fine to coarse grained, with cobbles
	2 --		
	3 --		
	4 --		
	5 --		
	6 --		grayish brown, dense
	7 --		
	8 --		
	9 --		
	10 --		
	11 --		
	12 --		
	13 --		
	14 --	SM	Silty Sand, dark brown, moist, dense, fine to medium grained
	15 --		
	16 --		
	17 --		
	18 --		
	19 --		
	20 --		
	21 --	SM/SW	Silty Sand to Sand, dark to yellowish brown, moist, dense, fine to coarse grained, with cobbles
	22 --		
	23 --		
	24 --		
	25 --		

BORING LOG NUMBER 13

Chandler Partners

File No. 22119

km

	Depth in feet	USCS Class.	Description
	26		
	27		
	28		
	29		
	30		
	31		
	32		**BEDROCK (SAUGUS FORMATION): Sandstone to conglomerate, yellow and grayish brown, moderately hard to hard, fine to coarse grained, with cobbles**
	33		**@33' Bedding [N25E, 10NW]**
	34		
	35		**@35' Bedding [N20E, 11NW]**
	36		
	37		
	38		
	39		
	40		
	41		**Total Depth 40 feet** **No Water** **Fill to 1 foot**
	42		
	43		**NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.**
	44		
	45		
	46		
	47		
	48		
	49		
	50		

BORING LOG NUMBER 14

Chandler Partners | **Date: 10/01/21** | **Elevation: 1,413.0'***

File No. 22119 | **Method: 24-inch Fly Auger**

km

*Reference: Topographic Survey provided by Alliance, not dated

	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
	0 --		TOP SOIL: Silty Sand, dark and grayish brown, moist, medium dense, fine to coarse grained
	1 --		
		SM/SW	OLD ALLUVIUM: Silty Sand to Sand, yellow to grayish brown, slightly moist, dense, fine to coarse grained
	2 --		
	3 --		
	4 --		
	5 --		
	6 --		
			dense
	7 --		
	8 --		
	9 --		
	10 --		
	11 --		
	12 --		
	13 --		
	14 --		
	15 --		
	16 --		
	17 --		
	18 --		
	19 --		
	20 --		
	21 --		
	22 --		
	23 --		
	24 --		
	25 --		

BORING LOG NUMBER 14

Chandler Partners

File No. 22119

km

	Depth in feet	USCS Class.	Description
	26		
	27		
	28		
	29		BEDROCK (SAUGUS FORMATION): Sandstone to conglomerate, yellow and grayish brown, moist, moderately hard to hard, fine to coarse grained
	30		
	31		
	32		
	33		@33' Bedding [N30E, 15NW]
	34		@34' Bedding [N40E, 15NW]
	35		
	36		
	37		
	38		
	39		
	40		
	41		Total Depth 40 feet No Water Fill to 1 foot
	42		
	43		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
	44		
	45		
	46		
	47		
	48		
	49		
	50		

LOG OF TEST PIT NUMBER 1

Chandler Partners | **Drilling Date: 04/26/21** | **Elevation: 1,371.0'***

File No. 22119 | **Method: Hand Dig**

dy/km

*Reference: Topographic Survey provided by Alliance, not dated

Sample Depth ft.	Moisture Content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground, Slope
			0		TOP SOIL: Silty Sand, dark brown, moist, medium dense, fine grained
1	7.3	102.6	1	SM	OLD ALLUVIUM: Silty Sand, dark brown, moist, dense, fine grained
			2		
3	8.4	91.7	3		
			4		
5	8.8	110.1	5	SP/SM	Silty Sand to Sand, dark brown, moist, dense, fine to medium grained, few cobbles
			6		
7	8.1	87.4	7		
			8		dense
			9		
10	8.9	96.1	10		Total Depth 10 feet No Water Fill to 1 foot NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual. Used 4-inch diameter Hand-Augering Equipment; Hand Sampler
			11		
			12		
			13		
			14		
			15		
			16		
			17		
			18		
			19		
			20		
			21		
			22		
			23		
			24		
			25		

LOG OF TEST PIT NUMBER 2

Chandler Partners | **Drilling Date: 04/26/21** | **Elevation: 1,409.0'***

File No. 22119 | **Method: Hand Dig**

dy/km

*Reference: Topographic Survey provided by Alliance, not dated

Sample Depth ft.	Moisture Content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
					Surface Conditions: Bare Ground
			0		TOP SOIL: Silty Sand, dark brown moist, medium dense, fine grained
			1		
2	7.0	103.6	2		
				SM	OLD ALLUVIUM: Silty Sand, dark brown, moist, dense, fine grained
			3		
4	5.6	112.2	4		
			5		
				SW	Sand, yellowish brown, moist, dense, fine to coarse grained, few cobbles
			6		
7	4.2	116.6	7		
			8		
			9		
10	5.7	114.7	10		
					Total Depth 10 feet No Water Fill to 2 feet
			11		
			12		
			13		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
			14		
					Used 4-inch diameter Hand-Augering Equipment; Hand Sampler
			15		
			16		
			17		
			18		
			19		
			20		
			21		
			22		
			23		
			24		
			25		

LOG OF TEST PIT NUMBER 3

Chandler Partners | **Drilling Date: 04/27/21** | **Elevation: 1,351.0'***

File No. 22119 | **Method: Hand Dig**

dy/km

*Reference: Topographic Survey provided by Alliance, not dated

Sample Depth ft.	Moisture Content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
			0		TOP SOIL: Silty Sand to Sand, dark brown, moist
1	4.7	95.6	1		
			2	SM/SP	OLD ALLUVIUM: Silty Sand to Sand, dark and yellowish brown, moist, dense, fine to medium grained, few cobbles
3	4.1	112.8	3		
			4	SW	Sand, yellowish brown, moist, dense, fine to coarse grained
5	6.4	88.3	5		
			6		
7	4.9	117.8	7		with cobbles
			8		
			9		
10	4.6	123.3	10		Total Depth 10 feet No Water Fill to 2 feet NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual. Used 4-inch diameter Hand-Augering Equipment; Hand Sampler
			11		
			12		
			13		
			14		
			15		
			16		
			17		
			18		
			19		
			20		
			21		
			22		
			23		
			24		
			25		

LOG OF TEST PIT NUMBER 4

Chandler Partners **Drilling Date: 04/27/21** **Elevation: 1,386.0'***

File No. 22119 **Method: Hand Dig**

dy/km *Reference: Topographic Survey provided by Alliance, not dated

Sample Depth ft.	Moisture Content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
			0 --		TOP SOIL: Silty Sand, dark brown, moist, medium dense, fine grained
			-		
			1 --		
			-		
2	3.7	123.7	2 --		
			-	SM/SW	OLD ALLUVIUM: Silty Sand to Sand, moist, dense, fine to coarse grained, few cobbles
			3 --		
			-		
4	5.1	106.1	4 --		
			-		
			5 --		
			-		yellowish brown, slightly moist
			6 --		
			-		
7	5.6	116.0	7 --		
			-		
			8 --		
			-		
			9 --		
			-		
10	6.7	118.1	10 --		
			-		Total Depth 10 feet No Water Fill to 2 feet
			11 --		
			-		
			12 --		
			-		
			13 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
			-		
			14 --		
			-		Used 4-inch diameter Hand-Augering Equipment; Hand Sampler
			15 --		
			-		
			16 --		
			-		
			17 --		
			-		
			18 --		
			-		
			19 --		
			-		
			20 --		
			-		
			21 --		
			-		
			22 --		
			-		
			23 --		
			-		
			24 --		
			-		
			25 --		
			-		

LOG OF TEST PIT NUMBER 5

Chandler Partners | **Drilling Date: 04/26/21** | **Elevation: 1,334.0'***

File No. 22119 | **Method: Backhoe**

dy/km

*Reference: Topographic Survey provided by Alliance, not dated

Sample Depth ft.	Moisture Content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
					Surface Conditions: Bare Ground
			0		FILL: Silty Sand, dark brown, moist, medium dense, fine grained
1	6.2	110.4	1		
			2		
3	5.0	109.2	3		
			4		
5	5.7	104.4	5		
				SM/SP	OLD ALLUVIUM: Silty Sand to Sand, dark brown, moist, dense, fine to medium grained, few cobbles
			6		
7	6.1	114.1	7		
					yellowish brown, medium dense to dense
			8		
			9		
10	5.7	112.8	10		
			11		
			12		
				SM/SW	Silty Sand to Sand, yellowish brown, slightly moist, dense, fine to coarse grained
13	9.3	104.8	13		
			14		
					Total Depth 14 feet No Water Fill to 5 feet
			15		
			16		
			17		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
			18		
					Used 4-inch diameter Hand-Augering Equipment; Hand Sampler
			19		
			20		
			21		
			22		
			23		
			24		
			25		

LOG OF TEST PIT NUMBER 6

Chandler Partners | **Drilling Date: 04/26/21** | **Elevation: 1,337.0'***

File No. 22119 | **Method: Backhoe**

dy/km

*Reference: Topographic Survey provided by Alliance, not dated

Sample Depth ft.	Moisture Content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
			0		TOP SOIL: Silty Sand, dark brown, moist, medium dense, fine grained
			1		
2	7.8	105.9	2	SM	OLD ALLUVIUM: Silty Sand, dark brown, moist, dense, fine grained
			3		
4	7.2	99.8	4		
			5		
			6		
7	2.3	114.1	7		
			8		
			9		
10	3.9	111.4	10	SM/SW	Silty Sand to Sand, dark and yellowish brown, moist, dense, fine to medium grained, few cobbles
			11		
12	3.3	119.7	12		yellowish brown, very dense
			13		
			14		Total Depth 14 feet No Water Fill to 2 feet
			15		
			16		
			17		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
			18		Used 4-inch diameter Hand-Augering Equipment; Hand Sampler
			19		
			20		
			21		
			22		
			23		
			24		
			25		

LOG OF TEST PIT NUMBER 7

Chandler Partners | **Drilling Date: 04/26/21** | **Elevation: 1,359.0'***

File No. 22119 | **Method: Backhoe**

dy/km

*Reference: Topographic Survey provided by Alliance, not dated

Sample Depth ft.	Moisture Content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
			0		TOP SOIL: Silty Sand, yellowish brown, moist, medium dense, fine grained
			1		
			2		
2.5	7.9	107.6			
			3	SM	OLD ALLUVIUM: Silty Sand, dark brown, moist, dense, fine grained
			4		
5	5.1	109.7	5	SM/SP	Silty Sand to Sand, dark and yellowish brown, moist, dense, fine grained, few cobbles
			6		
7	3.8	107.0	7		
			8		
			9		
10	4.3	115.6	10	SM/SW	Silty Sand to Sand, dark and yellowish brown, moist, dense, fine to coarse grained
			11		
12	5.6	114.5	12		
			13		Total Depth 13 feet No Water Fill to 2½ feet NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual. Used 4-inch diameter Hand-Augering Equipment; Hand Sampler
			14		
			15		
			16		
			17		
			18		
			19		
			20		
			21		
			22		
			23		
			24		
			25		

LOG OF TEST PIT NUMBER 8

Chandler Partners **Drilling Date: 04/26/21** **Elevation: 1,332.0'***

File No. 22119 **Method: Backhoe**

dy/km *Reference: Topographic Survey provided by Alliance, not dated

Sample Depth ft.	Moisture Content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
			0		FILL: Silty Sand, dark and yellowish brown, moist, medium dense, fine grained
			1		
			2		
2.5	6.3	114.5			
			3		
			4		
5	6.9	104.6	5	SM	OLD ALLUVIUM: Silty Sand, dark brown, moist, dense, fine grained, few cobbles
			6		
			7		
7.5	8.5	117.6			
			8		
			9		
10	9.4	99.2	10		
					Total Depth 10 feet No Water Fill to 4½ feet
			11		
			12		
			13		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
			14		
					Used 4-inch diameter Hand-Augering Equipment; Hand Sampler
			15		
			16		
			17		
			18		
			19		
			20		
			21		
			22		
			23		
			24		
			25		

LOG OF TEST PIT NUMBER 9

Chandler Partners | **Drilling Date: 04/26/21** | **Elevation: 1,328.0'***

File No. 22119 | **Method: Backhoe**

dy/km | *Reference: Topographic Survey provided by Alliance, not dated

Sample Depth ft.	Moisture Content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
			0		FILL: Silty Sand, dark brown, moist, medium dense, fine grained
			1		
			2		
				SM	OLD ALLUVIUM: Silty Sand, dark brown, moist, dense, fine grained, few cobbles
2.5	8.7	100.3	3		
			4		
5	6.9	109.5	5		
			6		
			7		
7.5	9.1	119.6			
			8		
			9		
10	8.3	108.8	10		
					Total Depth 10 feet No Water Fill to 2 feet
			11		
			12		
			13		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
			14		
			15		Used 4-inch diameter Hand-Augering Equipment; Hand Sampler
			16		
			17		
			18		
			19		
			20		
			21		
			22		
			23		
			24		
			25		

LOG OF TEST PIT NUMBER 10

Chandler Partners | **Drilling Date: 04/26/21** | **Elevation: 1,355.0'***

File No. 22119 | **Method: Backhoe**

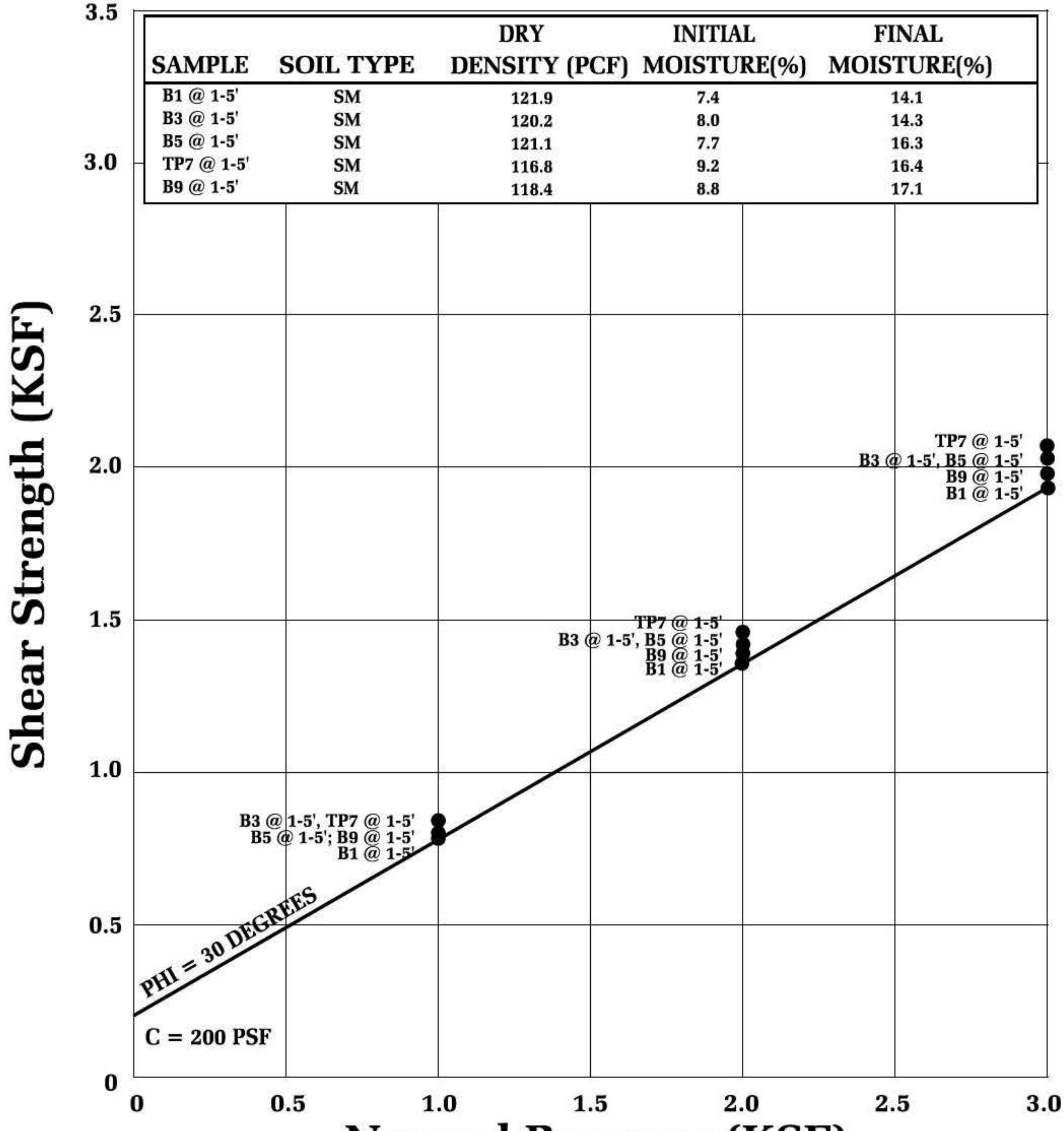
dy/km

*Reference: Topographic Survey provided by Alliance, not dated

Sample Depth ft.	Moisture Content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
			0		TOP SOIL: Silty Sand, dark brown, slightly moist, medium dense, fine to medium grained
			1		
			2		
2.5	5.7	104.9		SM	OLD ALLUVIUM: Silty Sand, dark brown, moist, dense, fine grained
			3		
			4		
5	6.1	110.5	5		
				SM/SW	Silty Sand to Sand, dark and yellowish brown, moist, dense, fine to coarse grained, few cobbles
			6		
7	5.4	117.7	7		
			8		
			9		
10	8.9	126.6	10		
					Total Depth 10 feet No Water Fill to 2 feet
			11		
			12		
			13		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
			14		Used 4-inch diameter Hand-Augering Equipment; Hand Sampler
			15		
			16		
			17		
			18		
			19		
			20		
			21		
			22		
			23		
			24		
			25		

BULK SAMPLE REMOLDED TO 90 PERCENT OF THE MAXIMUM LABORATORY DENSITY

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B1 @ 1-5'	SM	121.9	7.4	14.1
B3 @ 1-5'	SM	120.2	8.0	14.3
B5 @ 1-5'	SM	121.1	7.7	16.3
TP7 @ 1-5'	SM	116.8	9.2	16.4
B9 @ 1-5'	SM	118.4	8.8	17.1



● Direct Shear, Saturated

SHEAR TEST DIAGRAM

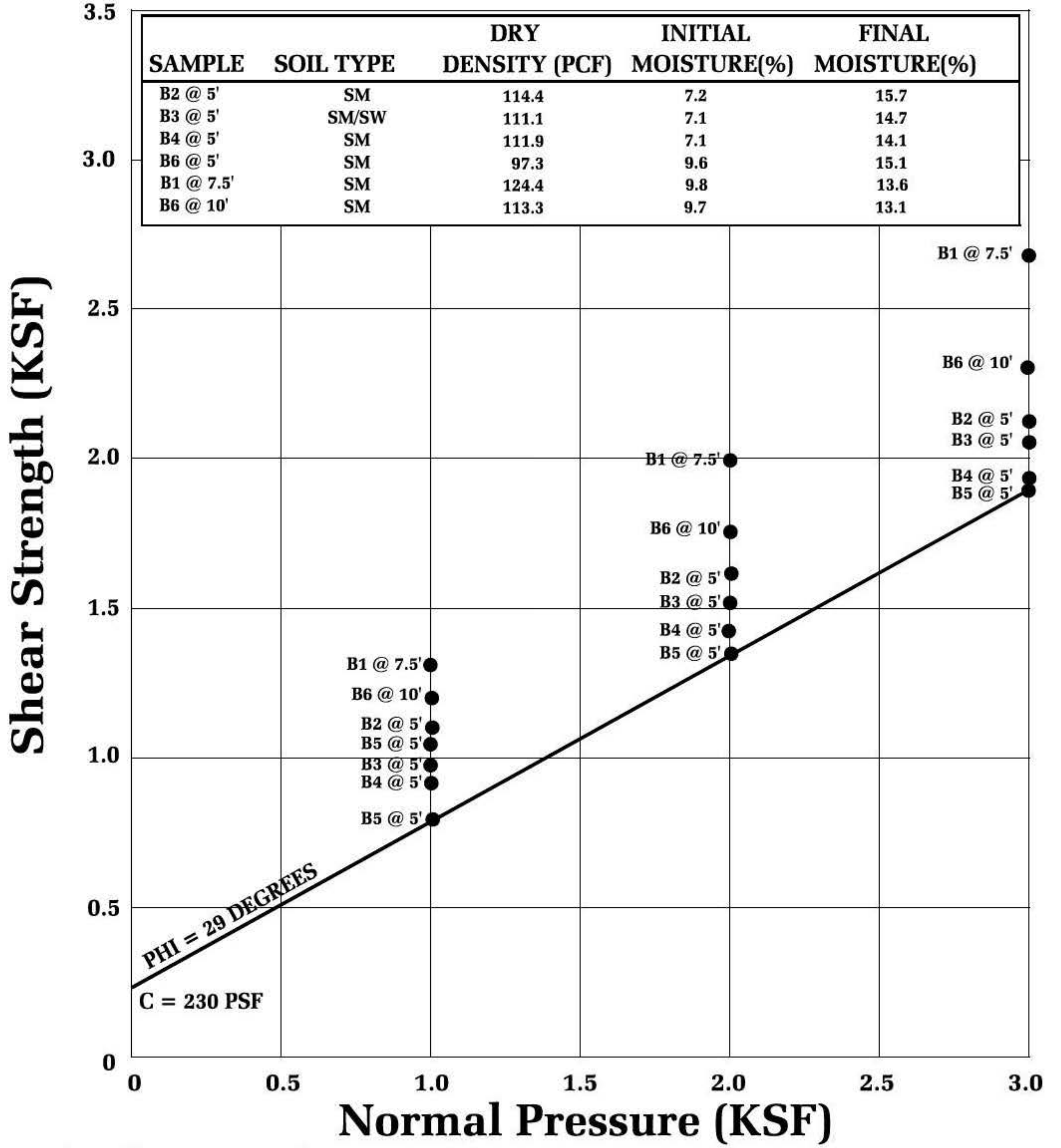
Geotechnologies, Inc.
Consulting Geotechnical Engineers

CHANDLER PARTNERS

FILE NO. 22119 | PLATE: B-1

ALLUVIUM

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B2 @ 5'	SM	114.4	7.2	15.7
B3 @ 5'	SM/SW	111.1	7.1	14.7
B4 @ 5'	SM	111.9	7.1	14.1
B6 @ 5'	SM	97.3	9.6	15.1
B1 @ 7.5'	SM	124.4	9.8	13.6
B6 @ 10'	SM	113.3	9.7	13.1



● Direct Shear, Saturated

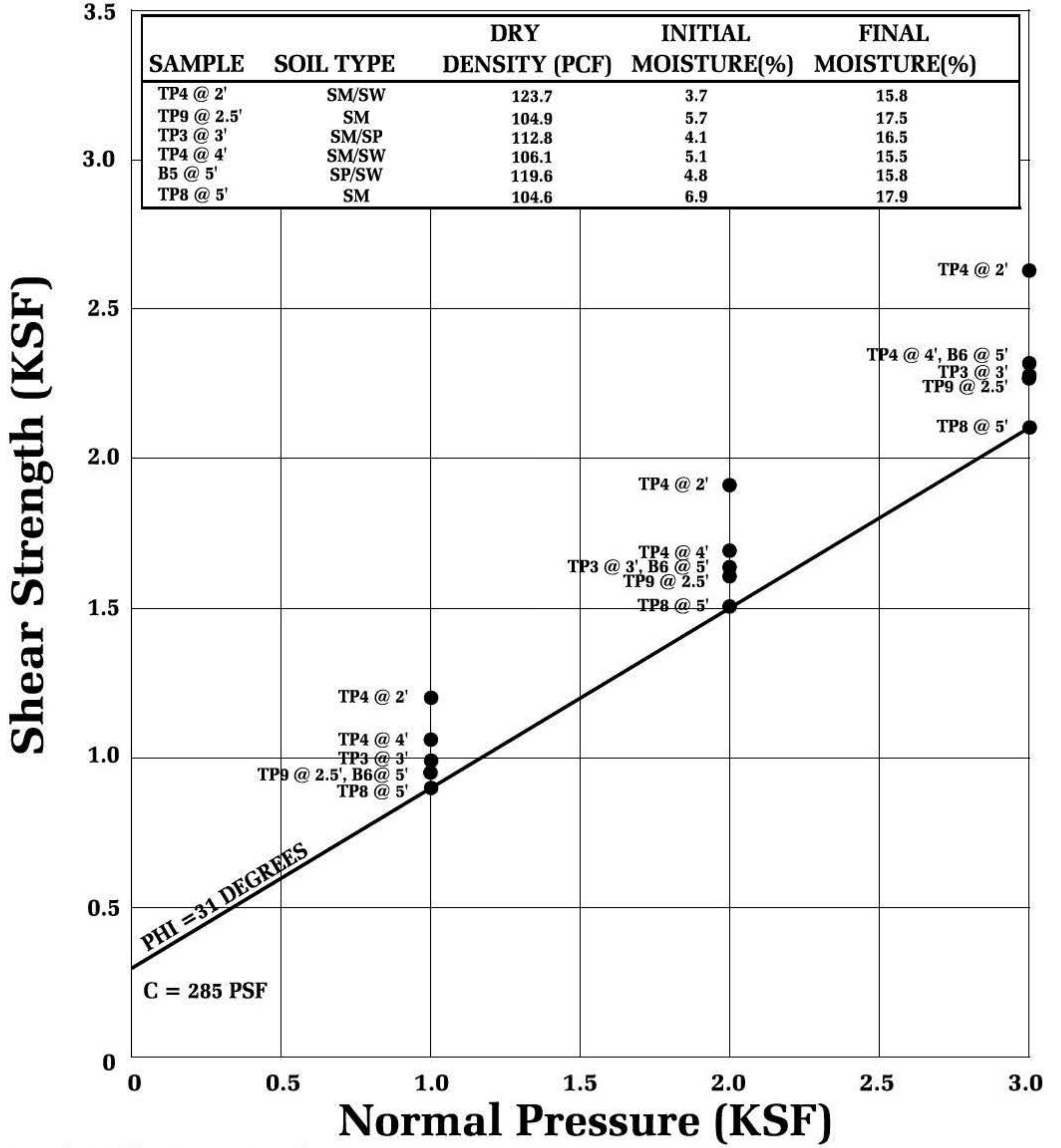
SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

CHANDLER PARTNERS	
FILE NO. 22119	PLATE: B-2

OLD ALLUVIUM

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
TP4 @ 2'	SM/SW	123.7	3.7	15.8
TP9 @ 2.5'	SM	104.9	5.7	17.5
TP3 @ 3'	SM/SP	112.8	4.1	16.5
TP4 @ 4'	SM/SW	106.1	5.1	15.5
B5 @ 5'	SP/SW	119.6	4.8	15.8
TP8 @ 5'	SM	104.6	6.9	17.9



● Direct Shear, Saturated

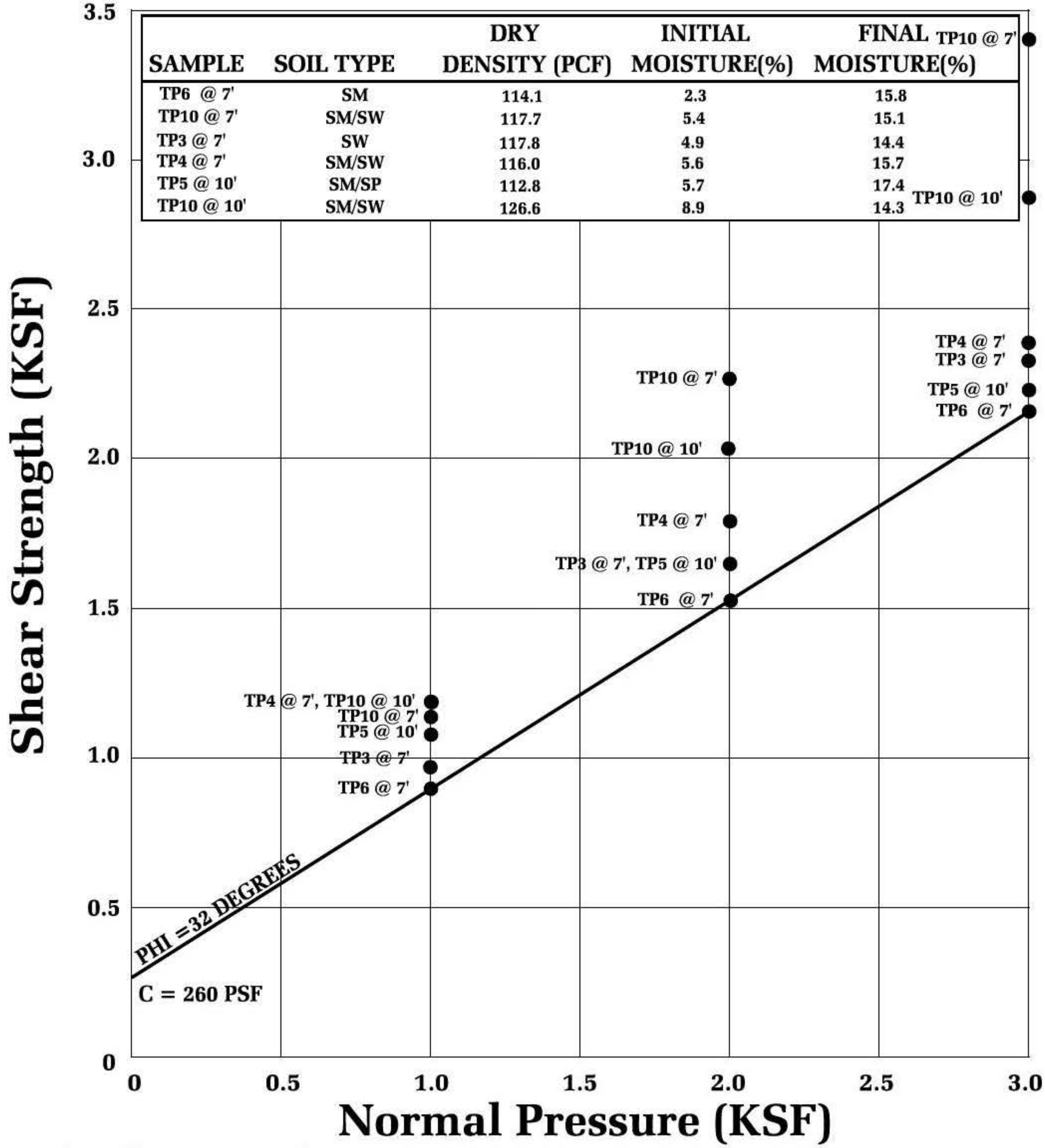
SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

CHANDLER PARTNERS

FILE NO. 22119 | PLATE: B3

OLD ALLUVIUM



SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
TP6 @ 7'	SM	114.1	2.3	15.8
TP10 @ 7'	SM/SW	117.7	5.4	15.1
TP3 @ 7'	SW	117.8	4.9	14.4
TP4 @ 7'	SM/SW	116.0	5.6	15.7
TP5 @ 10'	SM/SP	112.8	5.7	17.4
TP10 @ 10'	SM/SW	126.6	8.9	14.3

● Direct Shear, Saturated

SHEAR TEST DIAGRAM

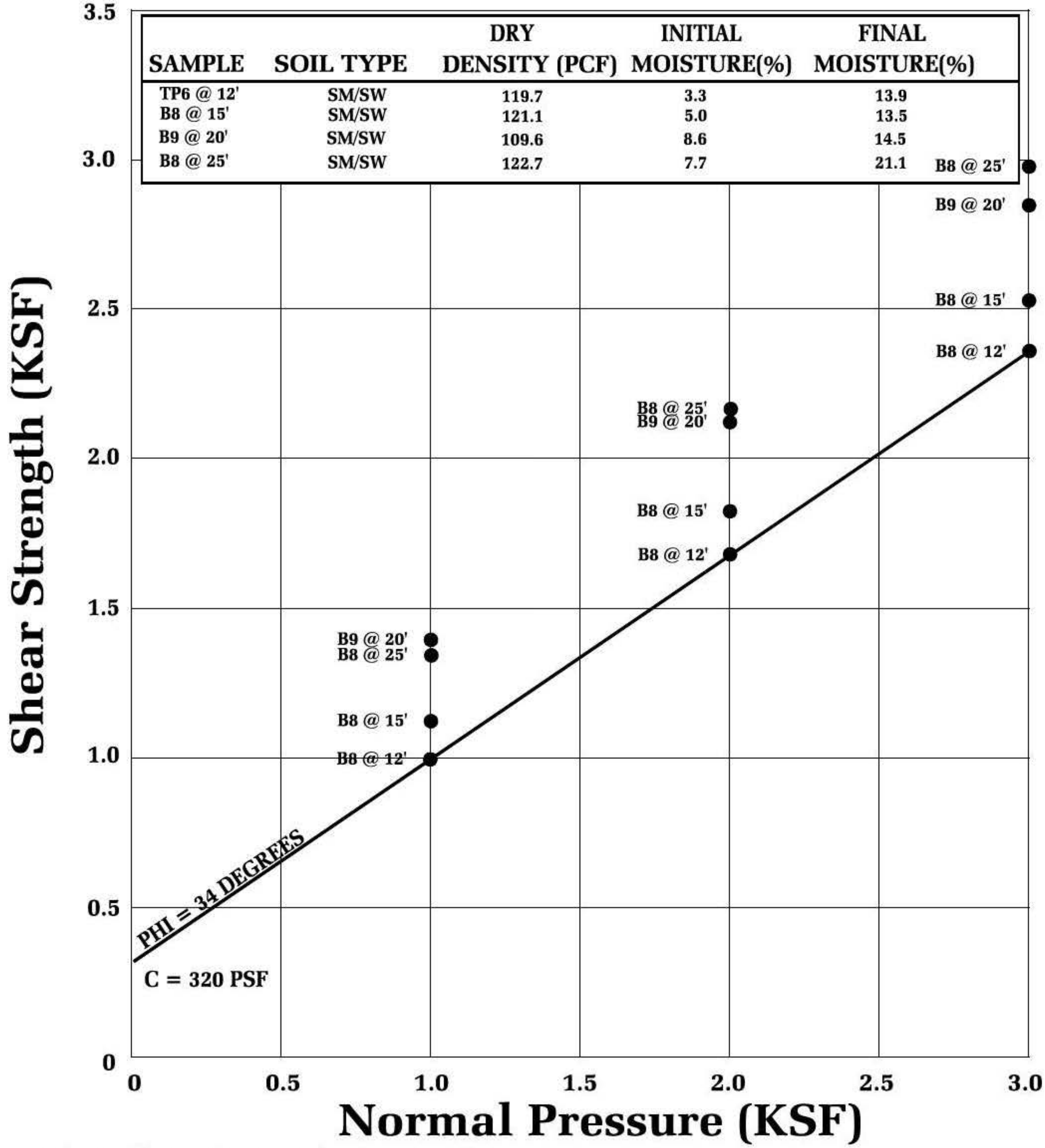
Geotechnologies, Inc.
Consulting Geotechnical Engineers

CHANDLER PARTNERS

FILE NO. 22119 | PLATE: B-4

OLD ALLUVIUM

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
TP6 @ 12'	SM/SW	119.7	3.3	13.9
B8 @ 15'	SM/SW	121.1	5.0	13.5
B9 @ 20'	SM/SW	109.6	8.6	14.5
B8 @ 25'	SM/SW	122.7	7.7	21.1



● Direct Shear, Saturated

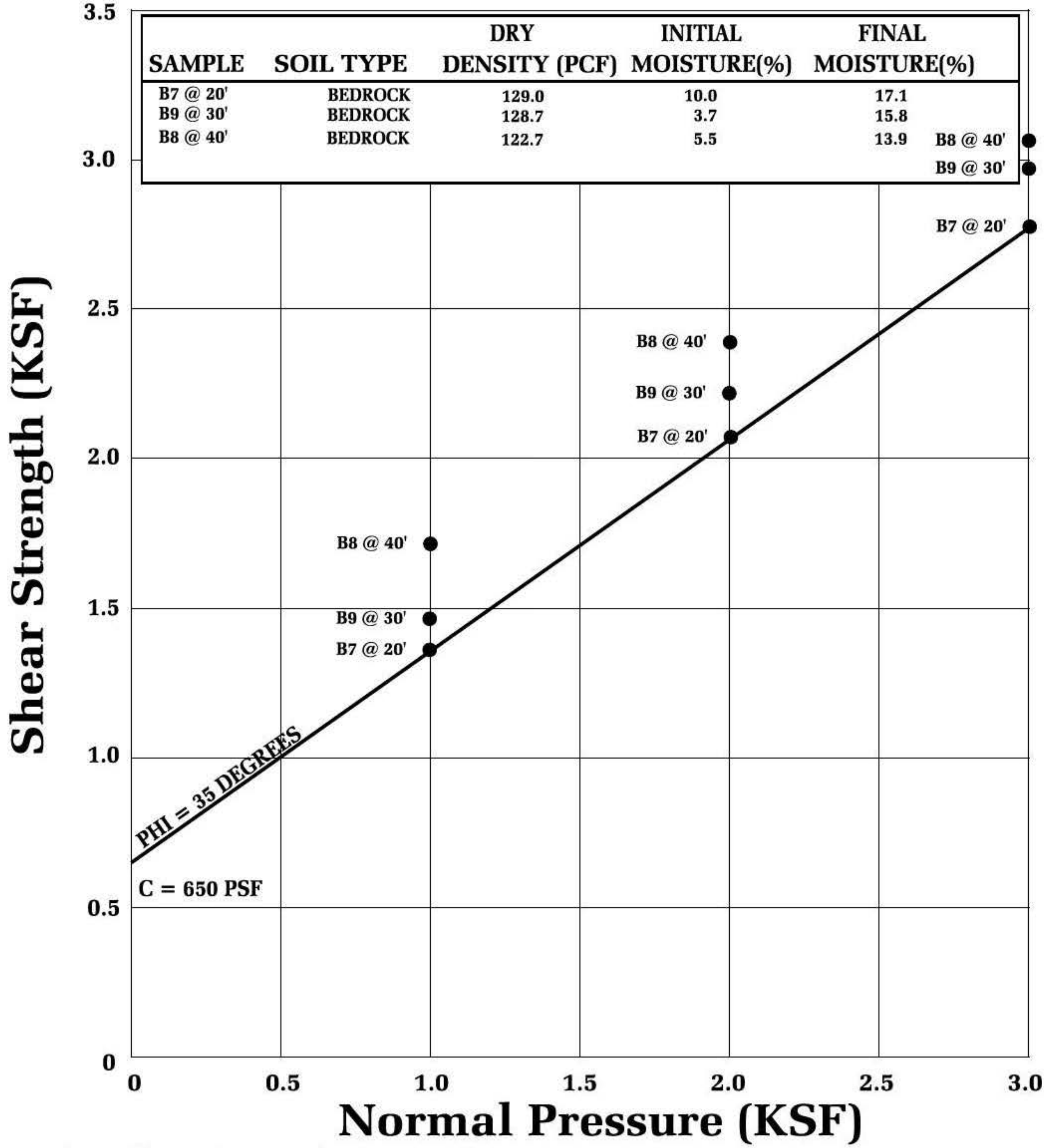
SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

CHANDLER PARTNERS

FILE NO. 22119 | PLATE: B-5

BEDROCK



SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B7 @ 20'	BEDROCK	129.0	10.0	17.1
B9 @ 30'	BEDROCK	128.7	3.7	15.8
B8 @ 40'	BEDROCK	122.7	5.5	13.9

● Direct Shear, Saturated

SHEAR TEST DIAGRAM

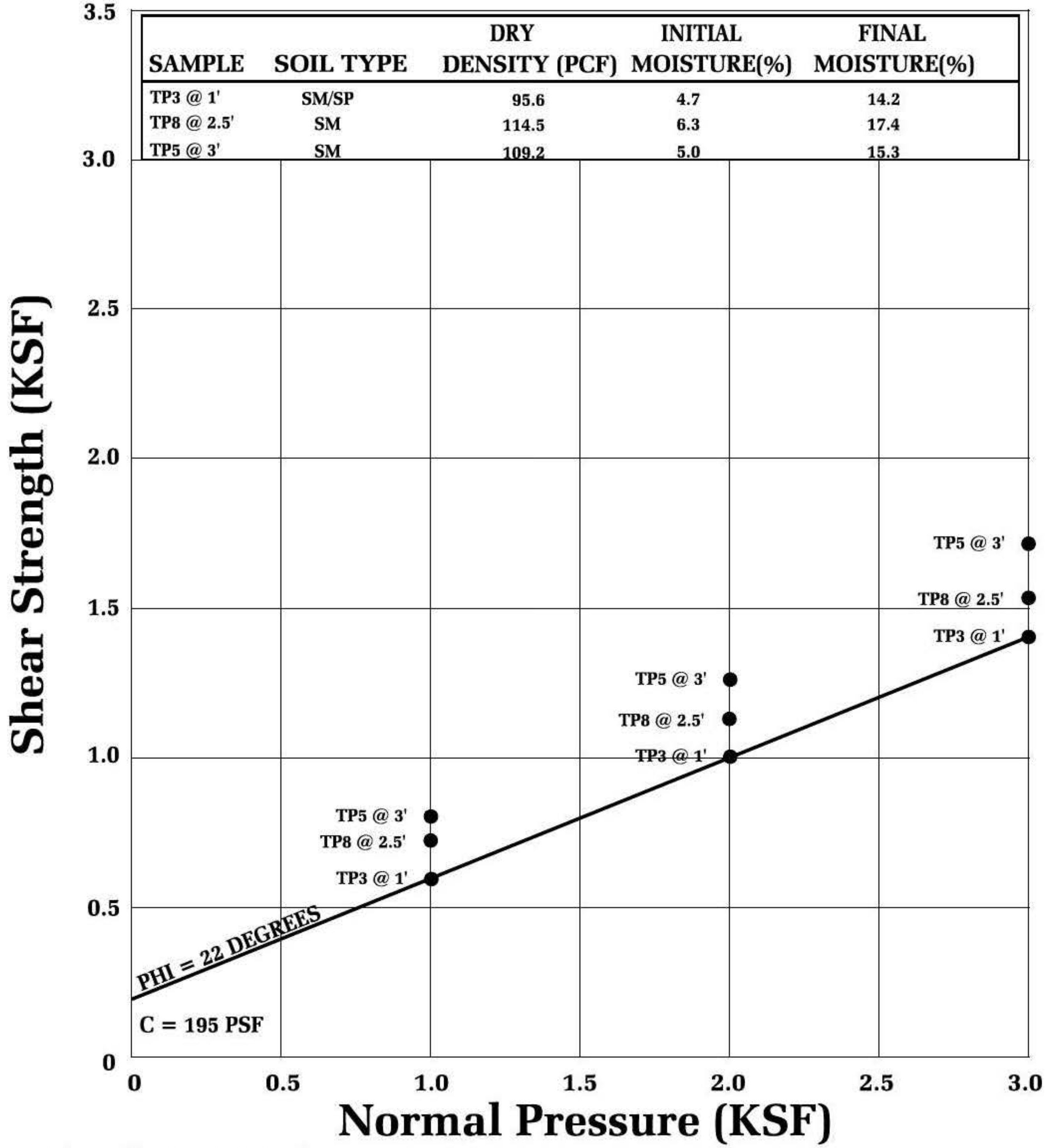
Geotechnologies, Inc.
Consulting Geotechnical Engineers

CHANDLER PARTNERS

FILE NO. 22119 | PLATE: B-6

EXISTING FILL

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
TP3 @ 1'	SM/SP	95.6	4.7	14.2
TP8 @ 2.5'	SM	114.5	6.3	17.4
TP5 @ 3'	SM	109.2	5.0	15.3



● Direct Shear, Saturated

SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

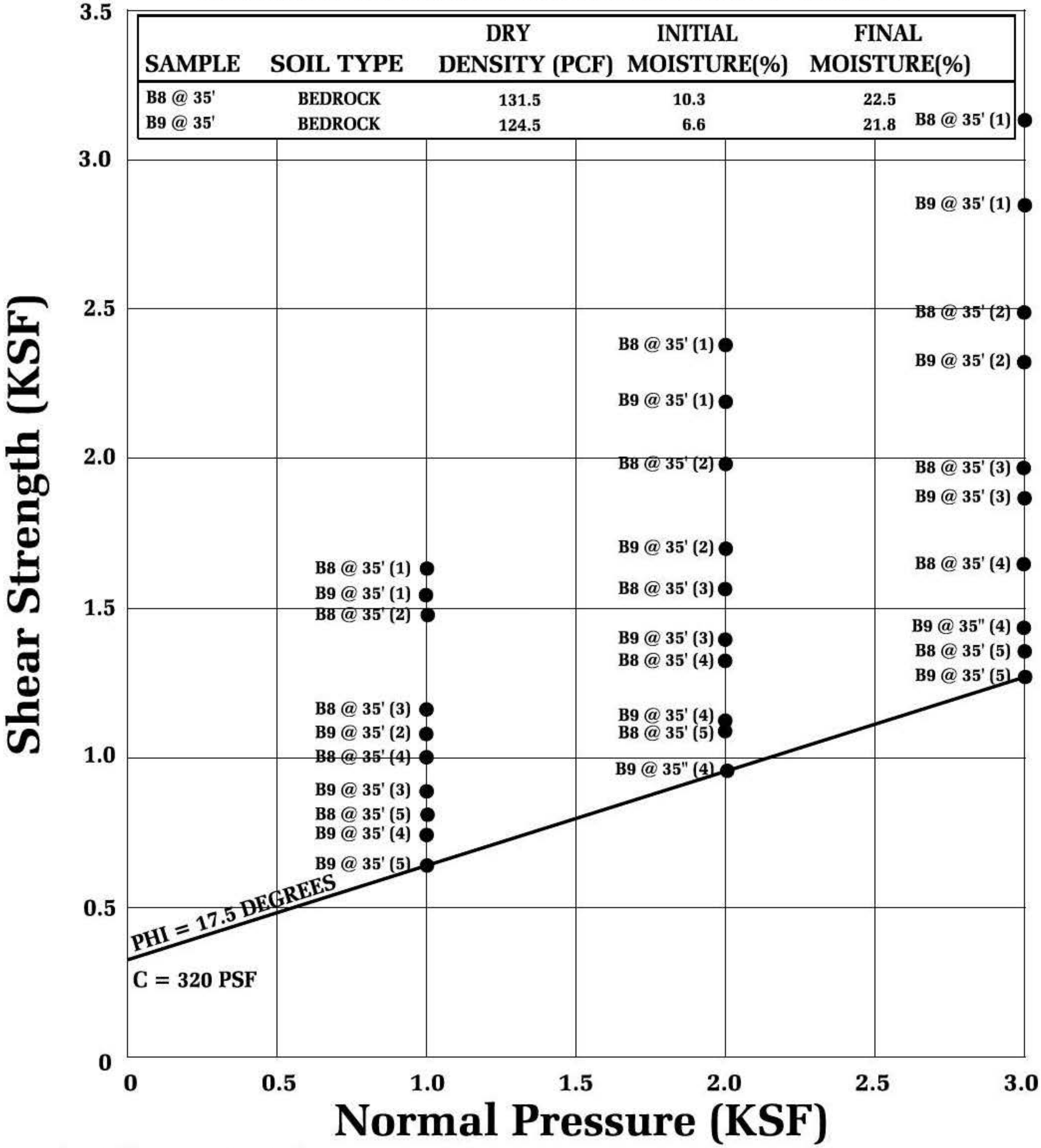
CHANDLER PARTNERS

FILE NO. 21864 | PLATE: B-7

BEDROCK (REPEAT SHEARING)

REMOLDED SAMPLE RESHEARED 5 TIMES

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B8 @ 35'	BEDROCK	131.5	10.3	22.5
B9 @ 35'	BEDROCK	124.5	6.6	21.8



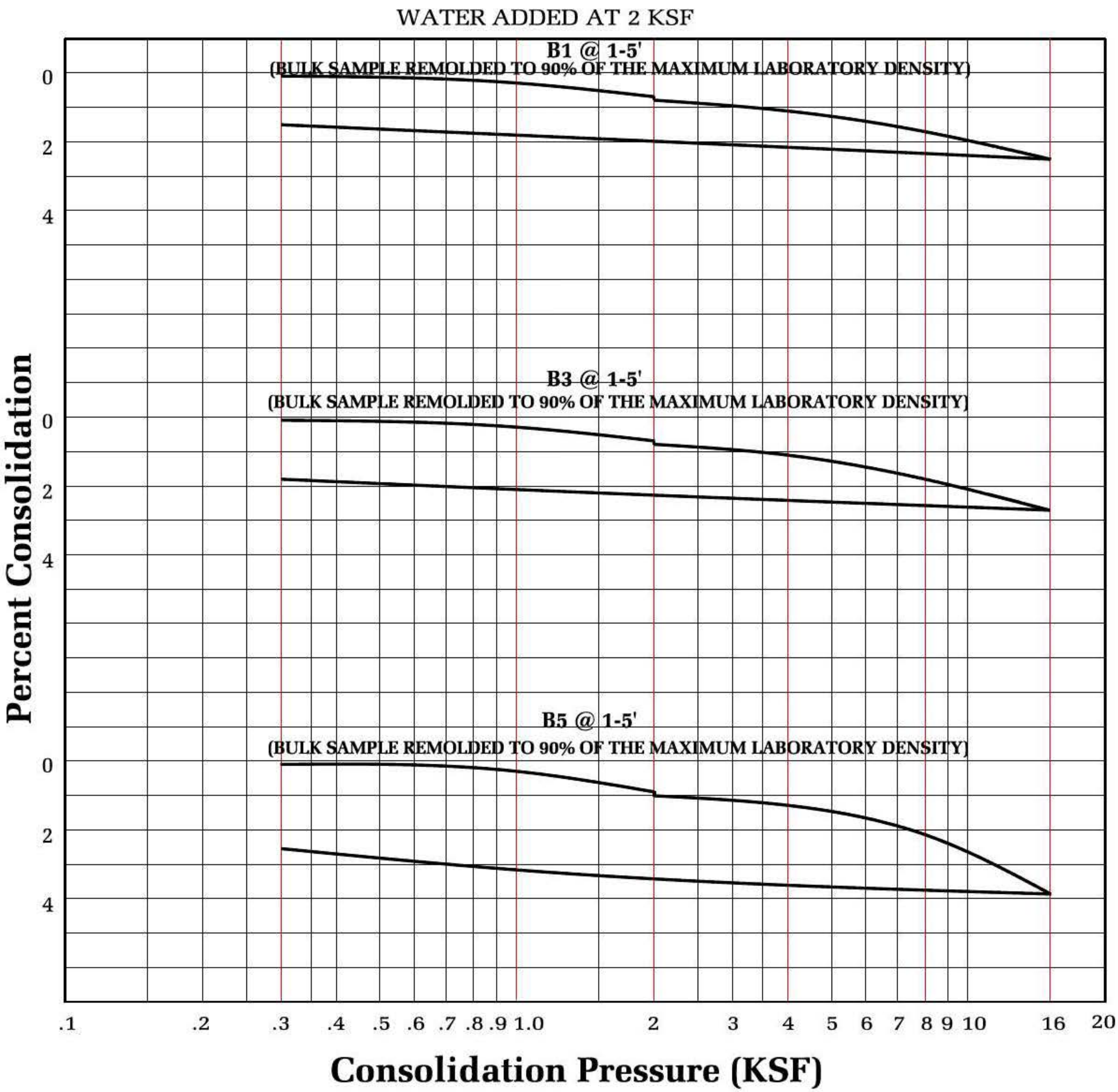
● Direct Shear, Saturated

SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

CHANDLER PARTNERS

FILE NO. 22119 | PLATE: B-8

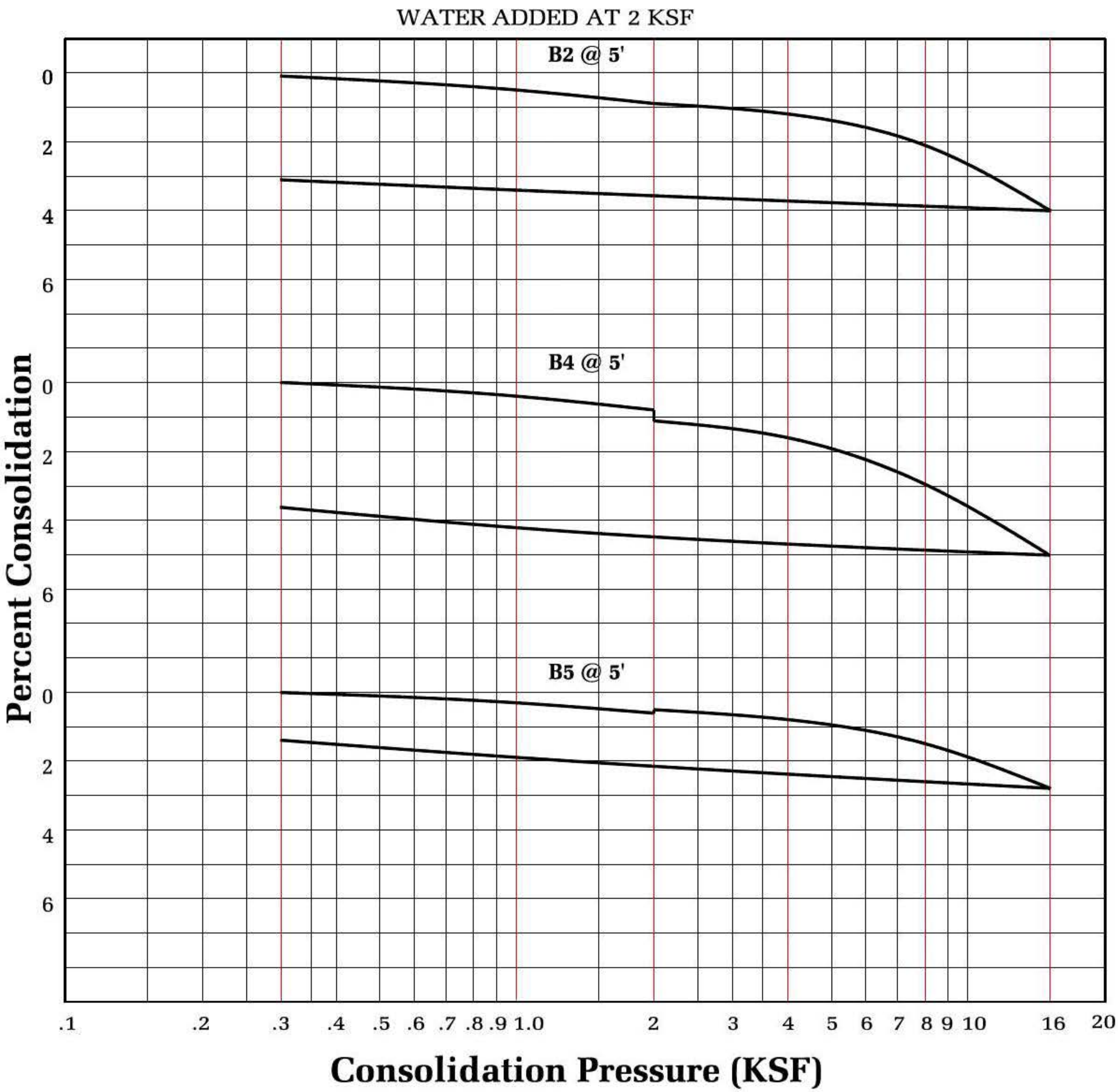


CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

CHANDLER PARTNERS

FILE NO. 22119 | **PLATE: C-1**



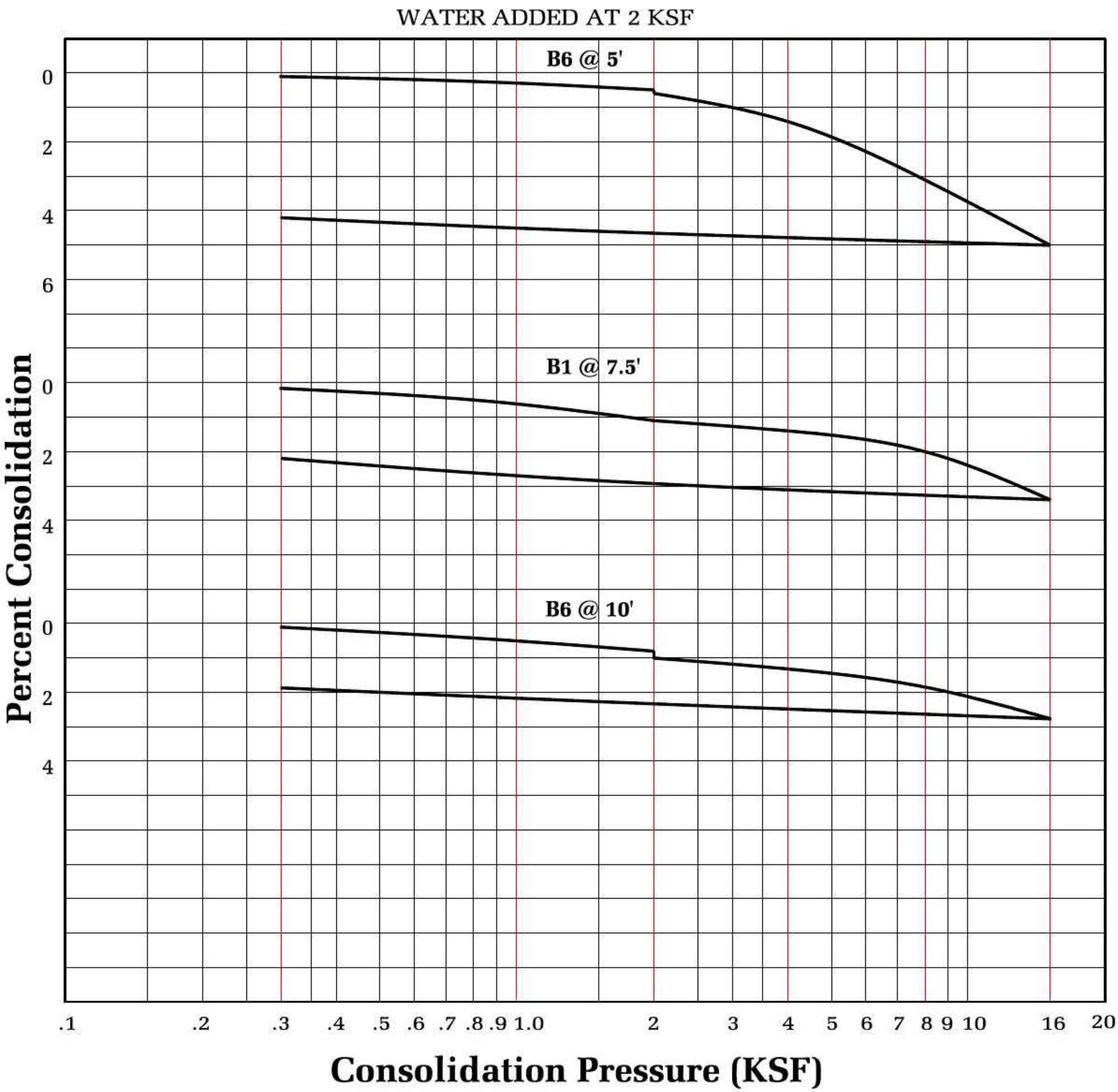
CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

CHANDLER PARTNERS

FILE NO. 22119

PLATE: C-2

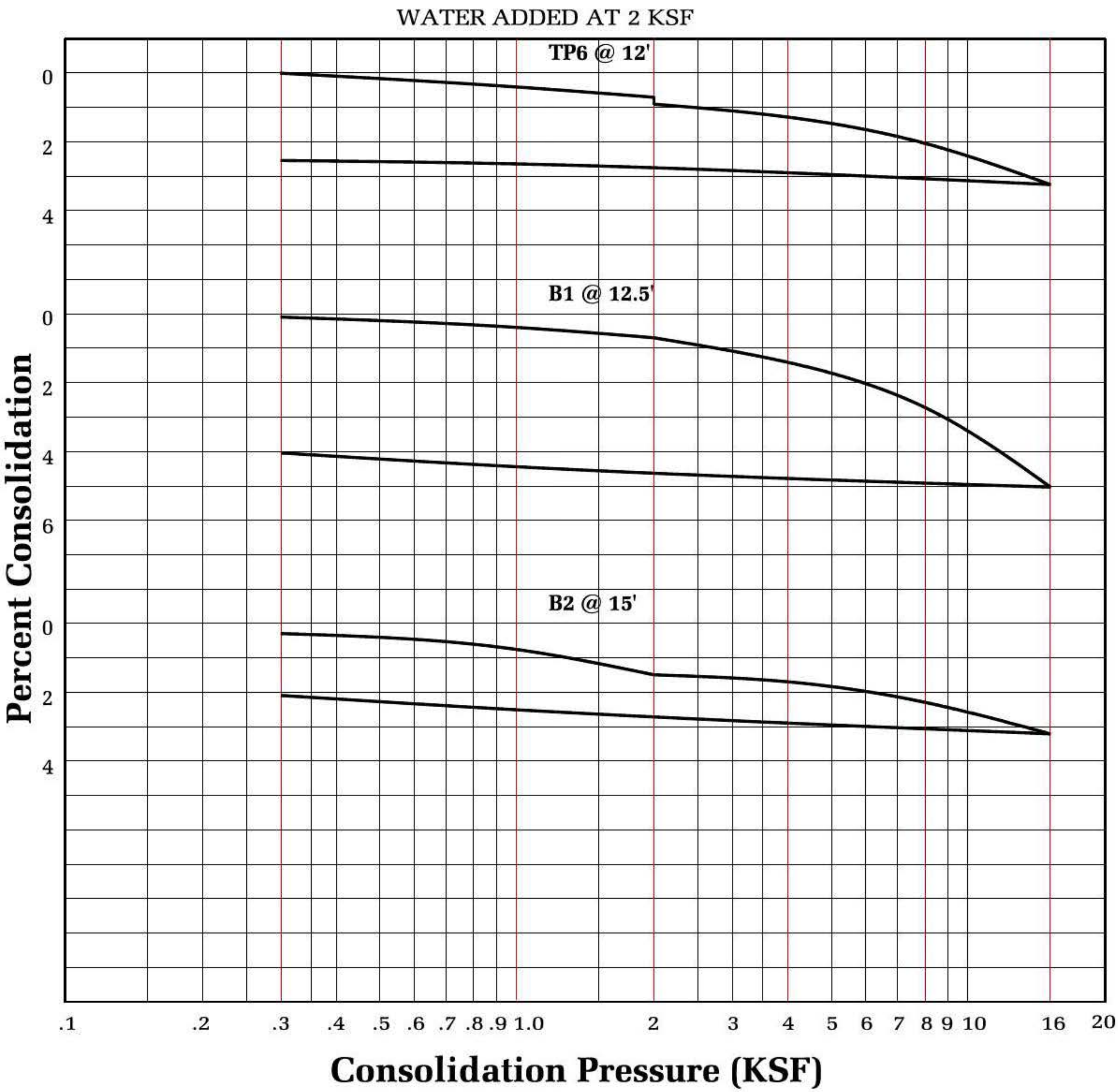


CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

CHANDLER PARTNERS

FILE NO. 22119 | **PLATE: C-3**



CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

CHANDLER PARTNERS

FILE NO. 22119 | **PLATE: C-4**

ASTM D-1557

SAMPLE	B1 @ 1- 5'	B3 @ 1-5'	B5 @ 1-5'	TP7 @ 1-5'	B9 @ 1-5'
SOIL TYPE:	SM	SM	SM	SM	SM
MAXIMUM DENSITY pcf.	135.4	133.6	134.5	129.8	131.5
OPTIMUM MOISTURE %	7.4	8.0	7.7	9.2	8.8

ASTM D 4829

SAMPLE	B1 @ 1-5'	B3 @ 1-5'	B5 @ 1-5'	TP7 @ 1-5'	B9 @ 1-5'
SOIL TYPE:	SM	SM	SM	SM	SM
EXPANSION INDEX UBC STANDARD 18-2	7	7	28	13	20
EXPANSION CHARACTER	VERY LOW	VERY LOW	LOW	VERY LOW	VERY LOW

SULFATE CONTENT

SAMPLE	B1 @ 1- 5'	B3 @ 1-5'	B5 @ 1-5'	TP7 @ 1-5'	B9 @ 1-5'
SULFATE CONTENT: (percentage by weight)	< 0.10 %	< 0.10 %	< 0.10 %	< 0.10 %	< 0.10 %

COMPACTION/EXPANSION/SULFATE DATA SHEET

Geotechnologies, Inc.
Consulting Geotechnical Engineers

CHANDLER PARTNERS

FILE NO. 22119 | PLATE: D

GEOTECHNOLOGIES, INC.

FILE NO.: 22119
PROJECT: Chandler Partners
BORING 1

EVALUATION OF EARTHQUAKE-INDUCED SETTLEMENTS IN DRY SANDY SOILS

INPUT:

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.7
Peak Horiz. Acceleration (g):	1.18

Depth of Base of Strata (ft)	Thickness of Layer (ft)	USCS Soil Type	Depth of Mid-point of Layer (ft)	Soil Unit Weight (pcf)	Overburden Pressure at Mid-point (tsf)	Mean Effective Pressure at Mid-point (tsf)	Average Cyclic Shear Stress [Tav]	Field SPT [N]	Correction Factor [Cer]	Relative Density [Dr] (%)	Correction Factor [Cn]	Corrected [N1]60	Maximum Shear Mod. [Gmax] (tsf)	[geff]*[Geff] / [Gmax]	[geff]	[geff]*100%	Volumetric Strain [E15} (%)	Number of Strain Cycles [Nc]	Corrected Vol. Strains [Ec]	Settlement [S] (inches)
5.0	5.0	SM	2.5	115.8	0.14	0.10	0.111	N/A	-	-	-	-	-	-	-		COMPACTED FILL PAD			0.00
10.0	5.0	SM	7.5	136.7	0.46	0.31	0.351	23	1.3	95.0	1.38	41.3	857.862	3.56E-04	4.00E-03	4.00E-01	9.00E-02	8.6310	0.0702	0.08
22.5	12.5	SM/SP	16.3	123.5	1.02	0.68	0.760	34	1.3	99.0	0.97	42.9	1291.547	4.47E-04	3.50E-03	3.50E-01	8.50E-02	8.6310	0.0663	0.20
45.0	22.5	SM/SP	33.8	130.8	2.14	1.43	1.473	42	1.3	91.0	0.72	39.3	1819.475	5.04E-04	1.70E-03	1.70E-01	4.90E-02	8.6310	0.0382	0.21
50.0	5.0	SM/SP	47.5	121.8	3.03	2.03	1.894	74	1.3	100.0	0.61	58.7	2473.682	4.31E-04	9.00E-04	9.00E-02	7.90E-03	8.6310	0.0062	0.01

Total Calculated Dynamic Dry Settlement (inches) 0.5

Project:	Chandler Partners
Date:	2/11/2022
File No.:	22119
Failure Configuration Description:	Old Alluvium/Bedrock

ESTIMATION OF PERMANENT SEISMIC DISPLACEMENT USING THE BRAY AND RATHJE (1998) PROCEDURE.

INPUT PARAMETERS:

Yield Acceleration, ky (g):	20		Normalized MHEA Sigma:	0
Vertical Thickness, H (m):	0		Mean Period Sigma:	0
Shear Wave Vel., Vs (m/s):	360		Significant Duration Sigma:	0
Earthquake Magnitude, M:	6.70	*	Normalized Displacement Sigm	0
Earthquake Accel., Firm Rock, MHAr, (g):	0.67	*		
Earthquake Distance, r (km):	8.6	*	Allowable screen displacement	15 *
Landslide factor, 0.8 for large slide, 1.0 for small slid	1	*		

CALCULATIONS:

Site Period (Ts): 0.000

(eqn. 11.5) Ts = 4*H/Vs

NRF Factor: 0.826

(eqn. 11.3) NRF= 0.6225+ 0.9196*exp(-MHAr/0.4449)

Mean Period (Tm): 0.487

(eqn. 10.2a) Tm = (C1+C2*(M-6)+C3*r)*EXP(e_T) for M<7.25

(eqn. 10.2b) Tm = (C1+ 1.25* C2+C3*r)*EXP(e_T) for 7.25<M<8.0

where: C1 = 0.411
C2= 0.0837
C3 = 0.00208
e_T = 0.437

Duration, $D_{5\text{-}95}$ (s): 10.761

(eqn. 10.1a) $\ln(D_{5\text{-}95})$ = ln(exp(5.204+0.851*(M-6))/power(10,1.5*M+16.05), -1/3)/(15.7*10^6))+0.063*(r-10))+0.8664 for r>10 km

(eqn. 10.1b) $\ln(D_{5\text{-}95})$ = ln(exp(5.204+0.851*(M-6))/power(10,1.5*M+16.05), -1/3)/(15.7*10^6)+0.8664) for r<10 km

Ts/Tm:	0.0000
MHEA/MHA*NRF:	#NUM!
MHEA, kmax:	#NUM!
ky/kmax:	#NUM!
Normalized Disp. (cm/sec):	#NUM!

(Input values marked with asterisks are used for calculation of seismic coefficient for screen)

Estimated Displacement (cm):	#NUM!	**Median f_{eq} for Screen Procedure:**	0.349	*
Estimated Displacement (in):	#NUM!	**Seismic Coefficient for Screen Procedure (k_{eq}) :**	0.234	*

Geotechnologies, Inc.

Geotechnologies, Inc.

Project: Chandler Partners
File No.: 22119
Description: Cross Section A-A'

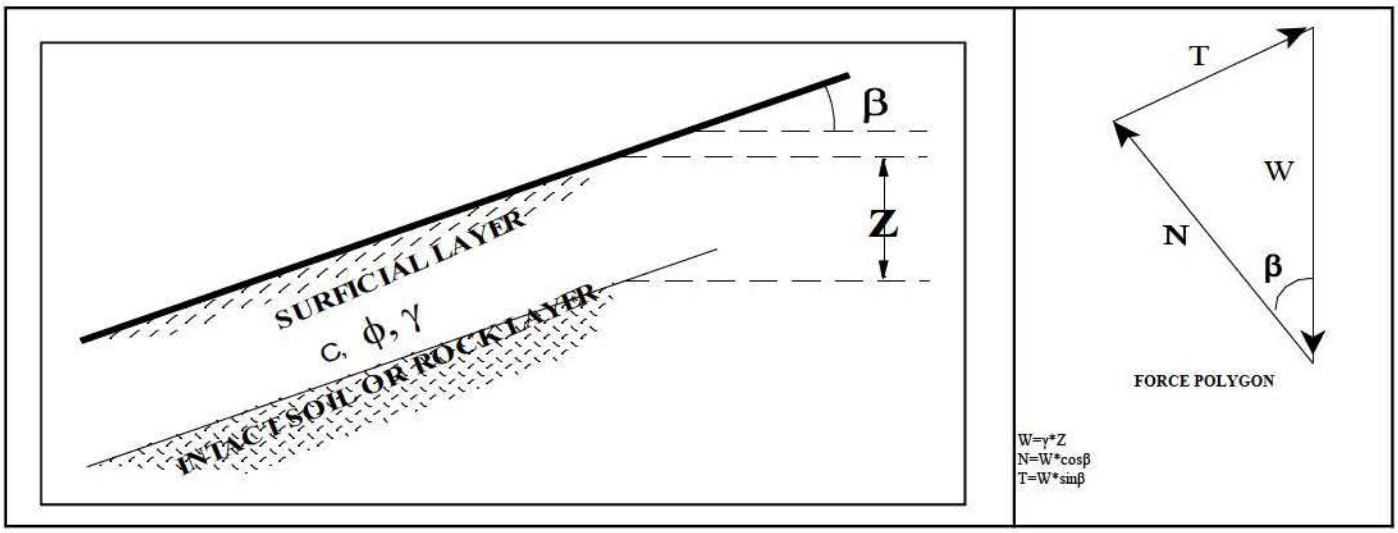
SURFICIAL SLOPE STABLITY FOR INFINITE SLOPE

Input Slope Properties:

Vertical Thickness of Surficial Materials	(Z)	4.0 feet	
Slope Angle	(β)	26.3 degrees	0.45902159 radians
Saturated Thickness	(h_s)	4.0 feet	

Input Soil Properties:

Unit Weight of Saturated Surficial Soils	(γ)	125.0 pcf	
Friction Angle of Surficial Soils	(ϕ)	31.0 degrees	0.54105207 radians
Cohesion of Surficial Soils	(c)	285.0 psf	
Density of Water	(γ_ω)	62.4 pcf	



Equation

$$F = \frac{c' + (\gamma - m*\gamma w)*z*\cos^2\beta*\tan\varphi}{\gamma*z*\sin\beta\cos\beta}$$

Factor of Safety	2.04

Ref: Blake, T.F., Hollingsworth, R.A., and Stewart, J.P., 2002, Recommended Procedures for Implementation of DMG Special Publication 117 Guidelines for analyzing and Mitigating Landslide Hazards in California, Southern California Earthquake Center

Geotechnologies, Inc.

Project: Chandler Partners
File No.: 22119
Description: Cross Section C-C'

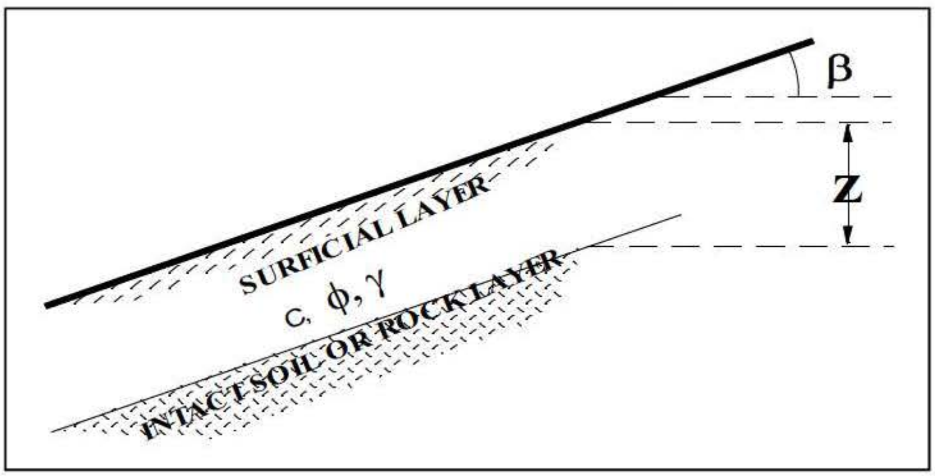
SURFICIAL SLOPE STABLITY FOR INFINITE SLOPE

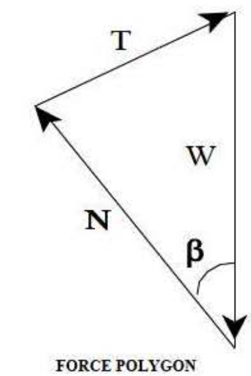
Input Slope Properties:

Vertical Thickness of Surficial Materials	(Z)	4.0 feet	
Slope Angle	(β)	26.3 degrees	0.45902159 radians
Saturated Thickness	(h_s)	4.0 feet	

Input Soil Properties:

Unit Weight of Saturated Surficial Soils	(γ)	120.0 pcf	
Friction Angle of Surficial Soils	(ϕ)	35.0 degrees	0.61086524 radians
Cohesion of Surficial Soils	(c)	650.0 psf	
Density of Water	(γ_ω)	62.4 pcf	





$W=\gamma*Z$
$N=W*\cos\beta$
$T=W*\sin\beta$

Equation $$F=\frac{c' + (\gamma - m*\gamma w)*z*\cos^{\wedge}2\beta*\tan\varphi}{\gamma*z*\sin\beta\cos\beta}$$

Factor of Safety	**4.09**

Ref: Blake, T.F., Hollingsworth, R.A., and Stewart, J.P., 2002, Recommended Procedures for Implementation of DMG Special Publication 117 Guidelines for analyzing and Mitigating Landslide Hazards in California, Southern California Earthquake Center

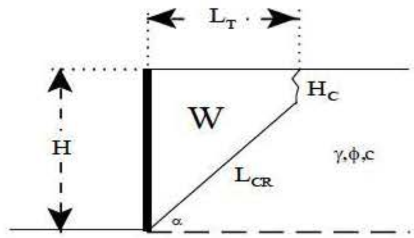
Geotechnologies, Inc.

Project: Chandler Partners
File No.: 22119
Description: Drained Catilever Retaining Wall (up to 11 feet)

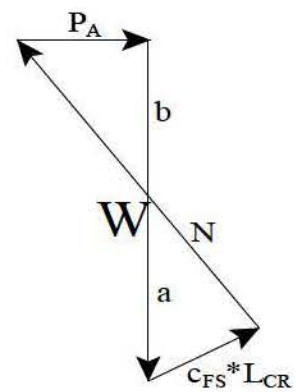
Retaining Wall Design with Level Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	11.00 feet
Unit Weight of Retained Soils	(γ)	120.0 pcf
Friction Angle of Retained Soils	(ϕ)	30.0 degrees
Cohesion of Retained Soils	(c)	200.0 psf
Factor of Safety	(FS)	1.50
Factored Parameters:	(ϕ_{FS})	21.1 degrees
	84.2	133.3 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	4.2	62	7409.5	10.6	4072.6	3336.9	1145.6
41	4.0	60	7232.2	10.6	3876.4	3355.8	1218.0
42	3.9	59	7048.0	10.6	3691.5	3356.5	1285.0
43	3.8	57	6859.5	10.6	3517.9	3341.6	1346.6
44	3.7	56	6668.7	10.5	3355.1	3313.6	1403.0
45	3.6	54	6476.9	10.4	3202.6	3274.3	1454.3
46	3.5	52	6285.2	10.4	3059.8	3225.4	1500.5
47	3.5	51	6094.5	10.3	2926.1	3168.4	1541.8
48	3.4	49	5905.2	10.2	2800.8	3104.4	1578.2
49	3.4	48	5717.8	10.1	2683.3	3034.5	1610.0
50	3.3	46	5532.6	10.0	2573.1	2959.5	1637.0
51	3.3	45	5349.7	9.9	2469.5	2880.3	1659.5
52	3.3	43	5169.3	9.8	2372.0	2797.3	1677.4
53	3.3	42	4991.4	9.7	2280.1	2711.3	1690.8
54	3.2	40	4816.0	9.6	2193.5	2622.6	1699.7
55	3.2	39	4643.2	9.5	2111.6	2531.6	1704.3
56	3.2	37	4472.8	9.4	2034.0	2438.8	1704.4
57	3.2	36	4304.9	9.2	1960.4	2344.4	1700.1
58	3.3	34	4139.2	9.1	1890.5	2248.7	1691.4
59	3.3	33	3975.8	9.0	1823.8	2152.0	1678.2
60	3.3	32	3814.5	8.9	1760.2	2054.3	1660.5
61	3.3	30	3655.2	8.8	1699.2	1956.0	1638.3
62	3.4	29	3497.8	8.6	1640.7	1857.2	1611.5
63	3.4	28	3342.2	8.5	1584.3	1758.0	1580.0
64	3.5	27	3188.2	8.4	1529.7	1658.5	1543.8
65	3.5	25	3035.7	8.2	1476.7	1558.9	1502.7



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+\phi_{FS})/\sin(\alpha-\phi_{FS})$

$b = W-a$

$P_A = b*\tan(\alpha-\phi_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\max}$ 1704.4 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 28.2 pcf

Design Wall for an Equivalent Fluid Pressure: 30 pcf

Geotechnologies, Inc.

Project: Chandler Partners

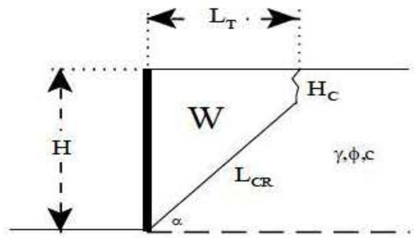
File No.: 22119

Description: Drained Catilever Retaining Wall (up to 14 feet)

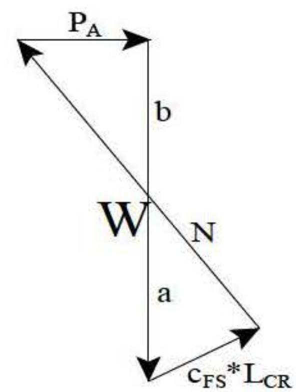
Retaining Wall Design with Level Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	14.00 feet
Unit Weight of Retained Soils	(γ)	120.0 pcf
Friction Angle of Retained Soils	(ϕ)	30.0 degrees
Cohesion of Retained Soils	(c)	200.0 psf
Factor of Safety	(FS)	1.50
Factored Parameters:	(ϕ_{FS})	21.1 degrees
	84.2	133.3 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	4.2	106	12772.4	15.3	5861.1	6911.3	2372.8
41	4.0	103	12408.9	15.2	5544.2	6864.7	2491.5
42	3.9	100	12045.8	15.1	5252.0	6793.8	2600.9
43	3.8	97	11685.2	15.0	4982.4	6702.8	2701.1
44	3.7	94	11328.5	14.8	4733.4	6595.2	2792.5
45	3.6	91	10976.9	14.7	4503.2	6473.7	2875.3
46	3.5	89	10630.8	14.5	4290.1	6340.7	2949.7
47	3.5	86	10290.8	14.4	4092.6	6198.2	3016.1
48	3.4	83	9957.0	14.2	3909.2	6047.8	3074.6
49	3.4	80	9629.6	14.1	3738.7	5890.9	3125.4
50	3.3	78	9308.5	13.9	3579.9	5728.7	3168.7
51	3.3	75	8993.7	13.8	3431.7	5562.1	3204.6
52	3.3	72	8685.1	13.6	3293.2	5391.9	3233.2
53	3.3	70	8382.4	13.5	3163.5	5218.9	3254.6
54	3.2	67	8085.5	13.3	3041.9	5043.6	3268.9
55	3.2	65	7794.1	13.1	2927.6	4866.5	3276.1
56	3.2	63	7508.1	13.0	2820.1	4688.0	3276.3
57	3.2	60	7227.2	12.8	2718.7	4508.5	3269.4
58	3.3	58	6951.1	12.7	2622.8	4328.3	3255.5
59	3.3	56	6679.7	12.5	2532.0	4147.6	3234.5
60	3.3	53	6412.6	12.4	2445.9	3966.7	3206.2
61	3.3	51	6149.6	12.2	2364.0	3785.7	3170.7
62	3.4	49	5890.5	12.0	2285.8	3604.7	3127.8
63	3.4	47	5635.1	11.9	2211.0	3424.0	3077.4
64	3.5	45	5383.0	11.7	2139.3	3243.7	3019.3
65	3.5	43	5134.1	11.5	2070.2	3063.8	2953.4



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+\phi_{FS})/\sin(\alpha-\phi_{FS})$

$b = W-a$

$P_A = b*\tan(\alpha-\phi_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 3276.3 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 33.4 pcf

Design Wall for an Equivalent Fluid Pressure: 34 pcf

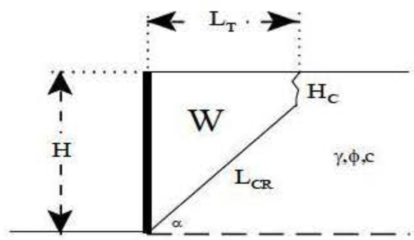
Geotechnologies, Inc.

Project: Chandler Partners
File No.: 22119
Description: Drained Catilever Retaining Wall (up to 18 feet)

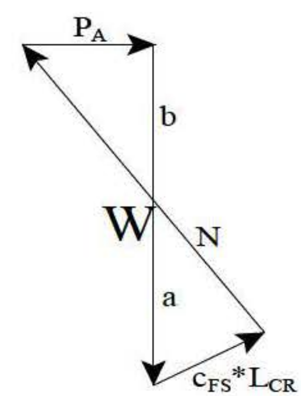
Retaining Wall Design with Level Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	18.00 feet
Unit Weight of Retained Soils	(γ)	120.0 pcf
Friction Angle of Retained Soils	(ϕ)	30.0 degrees
Cohesion of Retained Soils	(c)	200.0 psf
Factor of Safety	(FS)	1.50
Factored Parameters:	(ϕ_{FS})	21.1 degrees
	84.2	133.3 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	4.2	183	21925.1	21.5	8245.8	13679.3	4696.3
41	4.0	177	21243.7	21.3	7768.0	13475.8	4891.0
42	3.9	171	20575.3	21.1	7332.5	13242.8	5069.7
43	3.8	166	19921.0	20.8	6934.9	12986.0	5233.1
44	3.7	161	19281.4	20.6	6571.1	12710.3	5381.7
45	3.6	155	18656.9	20.3	6237.3	12419.5	5516.1
46	3.5	150	18047.3	20.1	5930.6	12116.7	5636.8
47	3.5	145	17452.5	19.9	5648.0	11804.5	5744.3
48	3.4	141	16872.1	19.6	5387.2	11485.0	5838.8
49	3.4	136	16305.7	19.4	5145.9	11159.8	5920.9
50	3.3	131	15752.8	19.1	4922.3	10830.6	5990.7
51	3.3	127	15212.9	18.9	4714.6	10498.3	6048.6
52	3.3	122	14685.4	18.7	4521.4	10164.0	6094.6
53	3.3	118	14169.7	18.5	4341.3	9828.4	6129.1
54	3.2	114	13665.3	18.2	4173.1	9492.3	6152.2
55	3.2	110	13171.7	18.0	4015.7	9156.0	6163.8
56	3.2	106	12688.3	17.8	3868.2	8820.2	6164.1
57	3.2	102	12214.6	17.6	3729.6	8485.0	6153.0
58	3.3	98	11750.1	17.4	3599.2	8150.9	6130.6
59	3.3	94	11294.3	17.2	3476.3	7818.0	6096.7
60	3.3	90	10846.6	17.0	3360.2	7486.4	6051.2
61	3.3	87	10406.7	16.8	3250.3	7156.5	5994.0
62	3.4	83	9974.1	16.6	3145.9	6828.1	5924.8
63	3.4	80	9548.2	16.4	3046.7	6501.5	5843.4
64	3.5	76	9128.8	16.2	2952.1	6176.7	5749.5
65	3.5	73	8715.3	16.0	2861.6	5853.7	5642.7



Design Equations (Vector Analysis):

$a = c_{FS} * L_{CR} * \sin(90+\phi_{FS})/\sin(\alpha-\phi_{FS})$

$b = W-a$

$P_A = b*\tan(\alpha-\phi_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 6164.1 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 38.1 pcf

Design Wall for an Equivalent Fluid Pressure: 39 pcf

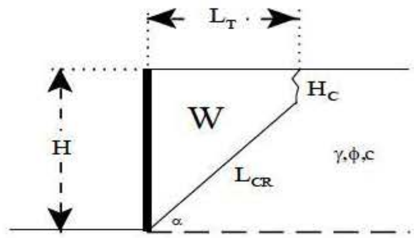
Geotechnologies, Inc.

Project: Chandler Partners
File No.: 22119
Description: Drained Catilever Retaining Wall (up to 20 feet)

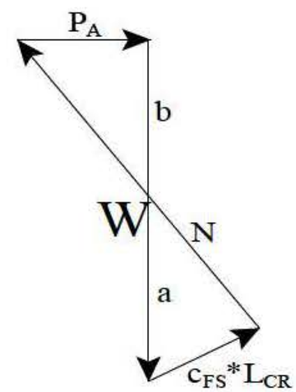
Retaining Wall Design with Level Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	20.00 feet
Unit Weight of Retained Soils	(γ)	120.0 pcf
Friction Angle of Retained Soils	(ϕ)	30.0 degrees
Cohesion of Retained Soils	(c)	200.0 psf
Factor of Safety	(FS)	1.50
Factored Parameters:	(ϕ_{FS})	21.1 degrees
	84.2	133.3 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	4.2	228	27359.5	24.6	9438.1	17921.3	6152.7
41	4.0	221	26489.4	24.3	8879.8	17609.6	6391.4
42	3.9	214	25639.7	24.1	8372.8	17266.9	6610.2
43	3.8	207	24811.0	23.8	7911.2	16899.7	6810.2
44	3.7	200	24003.4	23.5	7489.9	16513.5	6992.0
45	3.6	193	23216.9	23.2	7104.4	16112.5	7156.3
46	3.5	187	22450.8	22.9	6750.8	15700.0	7303.8
47	3.5	181	21704.8	22.6	6425.7	15279.1	7435.0
48	3.4	175	20978.0	22.3	6126.1	14851.8	7550.5
49	3.4	169	20269.7	22.0	5849.5	14420.2	7650.7
50	3.3	163	19579.1	21.8	5593.5	13985.6	7735.9
51	3.3	158	18905.5	21.5	5356.1	13549.4	7806.5
52	3.3	152	18248.0	21.2	5135.5	13112.5	7862.7
53	3.3	147	17605.9	21.0	4930.2	12675.7	7904.8
54	3.2	141	16978.4	20.7	4738.7	12239.7	7932.9
55	3.2	136	16364.7	20.5	4559.7	11805.0	7947.0
56	3.2	131	15764.1	20.2	4392.2	11371.9	7947.4
57	3.2	126	15175.9	20.0	4235.1	10940.9	7933.9
58	3.3	122	14599.5	19.7	4087.4	10512.1	7906.6
59	3.3	117	14034.2	19.5	3948.4	10085.8	7865.2
60	3.3	112	13479.4	19.3	3817.3	9662.0	7809.7
61	3.3	108	12934.4	19.1	3693.4	9241.0	7739.9
62	3.4	103	12398.7	18.8	3576.0	8822.6	7655.4
63	3.4	99	11871.7	18.6	3464.6	8407.1	7556.1
64	3.5	95	11352.9	18.4	3358.5	7994.4	7441.4
65	3.5	90	10841.7	18.2	3257.2	7584.4	7311.0



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+\phi_{FS})/\sin(\alpha-\phi_{FS})$

$b = W-a$

$P_A = b*\tan(\alpha-\phi_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\,max}$ 7947.4 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 39.7 pcf

Design Wall for an Equivalent Fluid Pressure: 40 pcf

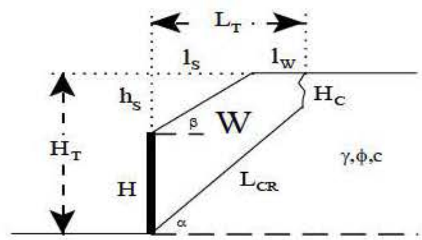
Geotechnologies, Inc.

Project: Chandler Partners
File No.: 22119
Description: Retaining Walls Up to 14 feet

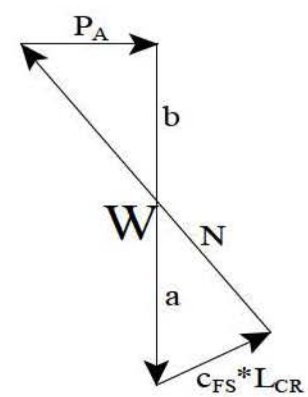
Retaining Wall Design with Transitioned Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	14.00 feet
Slope Angle of Backfill	(β)	26.3 degrees
Height of Slope above Wall	(h_s)	40.0 feet
Horizontal Length of Slope	(l_s)	80.9 feet
Total Height (Wall + Slope)	(H_T)	54.0 feet
Unit Weight of Retained Soils	(γ)	120.0 pcf
Friction Angle of Retained Soils	(ϕ)	30.0 degrees
Cohesion of Retained Soils	(c)	200.0 psf
Factor of Safety	(FS)	1.50
Factored Parameters:	(ϕ_{FS})	21.1 degrees
	(c_{FS})	133.3 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	4.2	239	28684.1	46.5	17822.3	10861.8	3729.0
41	4.0	220	26344.3	43.3	15800.4	10543.9	3826.9
42	3.9	202	24297.2	40.5	14112.7	10184.5	3898.9
43	3.8	187	22489.0	38.1	12688.8	9800.2	3949.2
44	3.7	174	20878.6	36.0	11476.0	9402.6	3981.2
45	3.6	162	19433.8	34.0	10434.2	8999.6	3997.2
46	3.5	151	18129.0	32.3	9532.4	8596.6	3999.2
47	3.5	141	16943.7	30.8	8746.4	8197.3	3988.9
48	3.4	132	15861.0	29.3	8056.9	7804.1	3967.5
49	3.4	124	14867.3	28.1	7448.5	7418.8	3936.0
50	3.3	116	13951.2	26.9	6908.9	7042.3	3895.3
51	3.3	109	13103.1	25.8	6427.9	6675.2	3845.9
52	3.3	103	12315.1	24.8	5997.2	6317.9	3788.4
53	3.3	97	11580.3	23.9	5609.8	5970.4	3723.2
54	3.2	91	10892.8	23.0	5260.0	5632.8	3650.7
55	3.2	85	10247.7	22.2	4942.9	5304.8	3571.1
56	3.2	80	9640.6	21.4	4654.5	4986.2	3484.6
57	3.2	76	9067.8	20.7	4391.1	4676.7	3391.4
58	3.3	71	8525.8	20.0	4149.8	4376.0	3291.4
59	3.3	67	8012.0	19.4	3928.1	4083.9	3184.8
60	3.3	63	7523.5	18.8	3723.6	3799.9	3071.5
61	3.3	59	7058.3	18.2	3534.5	3523.8	2951.4
62	3.4	55	6614.2	17.7	3358.9	3255.3	2824.6
63	3.4	52	6189.4	17.2	3195.5	2993.9	2690.9
64	3.5	48	5782.4	16.7	3042.8	2739.5	2550.0
65	3.5	45	5391.5	16.2	2899.7	2491.8	2402.0



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+\phi_{FS})/\sin(\alpha-\phi_{FS})$

$b = W-a$

$P_A = b*\tan(\alpha-\phi_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 3999.23 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 40.8 pcf

Design Wall for an Equivalent Fluid Pressure: 43 pcf

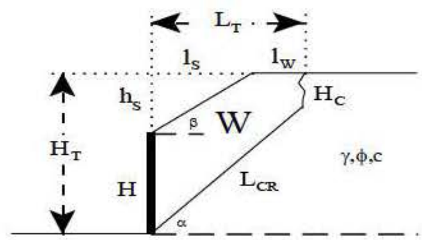
Geotechnologies, Inc.

Project: Chandler Partners
File No.: 22119
Description: Retaining Walls Up to 18 feet

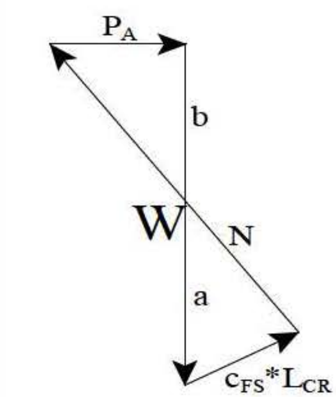
Retaining Wall Design with Transitioned Backfill (Vector Analysis)

Input:

Retaining Wall Height	(H)	18.00 feet
Slope Angle of Backfill	(β)	26.3 degrees
Height of Slope above Wall	(h_s)	40.0 feet
Horizontal Length of Slope	(l_s)	80.9 feet
Total Height (Wall + Slope)	(H_T)	58.0 feet
Unit Weight of Retained Soils	(γ)	120.0 pcf
Friction Angle of Retained Soils	(ϕ)	30.0 degrees
Cohesion of Retained Soils	(c)	200.0 psf
Factor of Safety	(FS)	1.50
Factored Parameters:	(ϕ_{FS})	21.1 degrees
	(c_{FS})	133.3 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	4.2	415	49761.1	61.6	23624.5	26136.6	8973.2
41	4.0	381	45709.4	57.5	20954.5	24754.9	8984.8
42	3.9	351	42165.1	53.8	18724.9	23440.2	8973.6
43	3.8	325	39035.6	50.6	16843.2	22192.3	8943.0
44	3.7	302	36249.7	47.8	15240.2	21009.5	8895.6
45	3.6	281	33751.6	45.2	13862.9	19888.6	8833.5
46	3.5	262	31496.9	43.0	12670.6	18826.3	8758.1
47	3.5	245	29450.0	40.9	11631.4	17818.6	8670.8
48	3.4	230	27581.9	39.0	10719.8	16862.0	8572.5
49	3.4	216	25868.6	37.3	9915.6	15952.9	8463.9
50	3.3	202	24290.4	35.8	9202.5	15087.9	8345.6
51	3.3	190	22830.7	34.4	8567.0	14263.7	8218.0
52	3.3	179	21475.6	33.1	7998.0	13477.5	8081.6
53	3.3	168	20213.1	31.8	7486.6	12726.5	7936.4
54	3.2	159	19033.2	30.7	7025.0	12008.2	7782.8
55	3.2	149	17927.1	29.7	6606.8	11320.3	7620.8
56	3.2	141	16887.3	28.7	6226.7	10660.6	7450.3
57	3.2	133	15907.1	27.7	5879.9	10027.2	7271.4
58	3.3	125	14980.9	26.9	5562.5	9418.4	7084.0
59	3.3	118	14103.7	26.1	5271.2	8832.5	6887.8
60	3.3	111	13270.8	25.3	5002.9	8267.9	6682.9
61	3.3	104	12478.5	24.5	4755.2	7723.4	6468.8
62	3.4	98	11723.3	23.8	4525.7	7197.6	6245.4
63	3.4	92	11001.8	23.2	4312.5	6689.4	6012.2
64	3.5	86	10311.5	22.5	4113.8	6197.7	5769.0
65	3.5	80	9649.6	21.9	3928.0	5721.6	5515.3



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+\phi_{FS})/\sin(\alpha-\phi_{FS})$

$b = W-a$

$P_A = b*\tan(\alpha-\phi_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\max}$ 8984.76 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 55.5 pcf

Design Wall for an Equivalent Fluid Pressure: 56 pcf

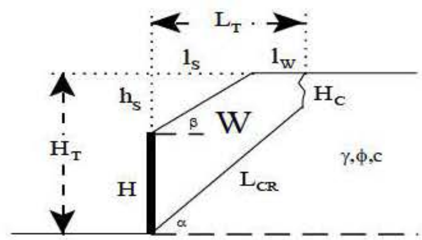
Geotechnologies, Inc.

Project: Chandler Partners
File No.: 22119
Description: Retaining Walls Up to 20 feet

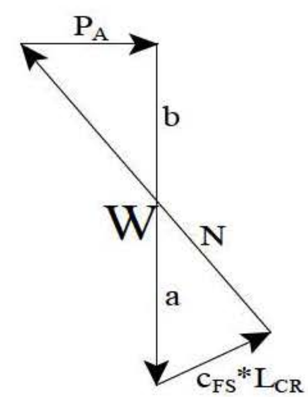
Retaining Wall Design with Transitioned Backfill (Vector Analysis)

Input:

Retaining Wall Height	(H)	20.00 feet
Slope Angle of Backfill	(β)	26.3 degrees
Height of Slope above Wall	(h_s)	40.0 feet
Horizontal Length of Slope	(l_s)	80.9 feet
Total Height (Wall + Slope)	(H_T)	60.0 feet
Unit Weight of Retained Soils	(γ)	120.0 pcf
Friction Angle of Retained Soils	(ϕ)	30.0 degrees
Cohesion of Retained Soils	(c)	200.0 psf
Factor of Safety	(FS)	1.50
Factored Parameters:	(ϕ_{FS})	21.1 degrees
	(c_{FS})	133.3 psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	4.2	520	62387.3	69.2	26525.6	35861.8	12312.0
41	4.0	478	57311.7	64.5	23531.5	33780.2	12260.4
42	3.9	441	52871.6	60.4	21031.0	31840.6	12189.5
43	3.8	408	48951.6	56.8	18920.5	30031.1	12101.8
44	3.7	379	45462.4	53.7	17122.3	28340.1	11999.5
45	3.6	353	42334.0	50.8	15577.3	26756.7	11883.9
46	3.5	329	39511.0	48.3	14239.8	25271.2	11756.4
47	3.5	308	36948.6	46.0	13073.9	23874.7	11617.8
48	3.4	288	34610.4	43.9	12051.3	22559.1	11468.8
49	3.4	271	32466.5	42.0	11149.2	21317.3	11310.0
50	3.3	254	30492.2	40.3	10349.3	20142.9	11141.6
51	3.3	239	28666.6	38.7	9636.5	19030.1	10964.1
52	3.3	225	26972.2	37.2	8998.5	17973.7	10777.6
53	3.3	212	25394.1	35.8	8425.0	16969.1	10582.2
54	3.2	199	23919.6	34.6	7907.5	16012.1	10377.8
55	3.2	188	22537.7	33.4	7438.8	15099.0	10164.6
56	3.2	177	21239.1	32.3	7012.7	14226.3	9942.3
57	3.2	167	20015.4	31.3	6624.3	13391.1	9710.7
58	3.3	157	18859.4	30.3	6268.9	12590.5	9469.8
59	3.3	148	17764.9	29.4	5942.7	11822.1	9219.3
60	3.3	139	16726.2	28.5	5642.6	11083.6	8958.7
61	3.3	131	15738.4	27.7	5365.5	10372.8	8687.9
62	3.4	123	14797.1	26.9	5109.1	9688.0	8406.3
63	3.4	116	13898.4	26.2	4871.0	9027.4	8113.6
64	3.5	109	13038.7	25.5	4649.2	8389.5	7809.2
65	3.5	102	12215.0	24.8	4442.2	7772.8	7492.5



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+\phi_{FS})/\sin(\alpha-\phi_{FS})$

$b = W-a$

$P_A = b*\tan(\alpha-\phi_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 12311.99 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 61.6 pcf

Design Wall for an Equivalent Fluid Pressure: 62 pcf

Geotechnologies, Inc.

Project: Chandler Partners

File No.: 22119

Seismically Induced Lateral Soil Pressure on Retaining Wall

(Ref: County of Los Angeles Department of Public Works, S004.0, Revised 1/6/202(

Input:

Height of Retaining Wall:	(H)	20.0 feet
Retained Soil Unit Weight:	(γ)	120.0 pcf
Peak Horizontal Ground Acceleration (S_{DS}/2.5):	(PGA)	0.68 g

Basement (Restrained) Walls with Level Backfill

$\Delta P_{AE} = (0.5*\gamma*H^2)*(0.68*PGA)$

$\Delta P_{AE} =$ 11097.6 lbs/ft

$EFP = 2*\Delta P_{AE}/H^2$

EFP = 55 pcf

Cantilevered (Unrestrained) Walls with Level Backfill

$\Delta P_{AE} = (0.5*\gamma*H^2)*(0.42*PGA)$

$\Delta P_{AE} =$ 6854.4 lbs/ft

$EFP = 2*\Delta P_{AE}/H^2$

EFP = 34 pcf

triangular distribution of pressure, applied to the proposed retaining wall.

Geotechnologies, Inc.

Project: Chandler Partners

File No.: 22119

Seismically Induced Lateral Soil Pressure on Retaining Wall

(Ref: County of Los Angeles Department of Public Works, S004.0, Revised 1/6/202(

Input:

Height of Retaining Wall:	(H)	20.0 feet
Retained Soil Unit Weight:	(γ)	120.0 pcf
Peak Horizontal Ground Acceleration (S_{DS}/2.5):	(PGA)	0.68 g

Cantilevered (Unrestrained) Walls with Sloping Backfill

$\Delta P_{AE} = (0.5*\gamma*H^2)*(0.70*PGA)$

$\Delta P_{AE} =$ 11424.0 lbs/ft

$EFP = 2*\Delta P_{AE}/H^2$

EFP = 57 pcf

triangular distribution of pressure, applied to the proposed retaining wall.

Geotechnologies, Inc.

Project: Chandler Partners
File No.: 22119
Date: 15-Feb-22

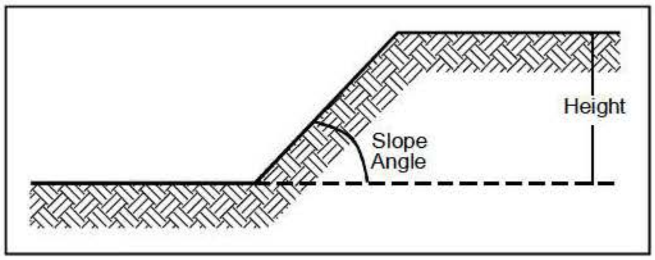
Slope Stability Calculations

Input

Soil Density	(γ)	120 pcf
Friction Angle	(ϕ)	31 degrees
Cohesion	(c)	285 psf
Factor of Safety	(FS)	1.5

Stability Number (N)

(ϕ_d)	21.8 degrees
$N_{(1.5:1)}$	0.030
$N_{(1:1)}$	0.058
$N_{(3/4:1)}$	0.076
$N_{(1:1.5)}$	0.083
$N_{(1:2)}$	0.100
$N_{(vertical)}$	0.176



Reference: Taylor's Chart (1937)

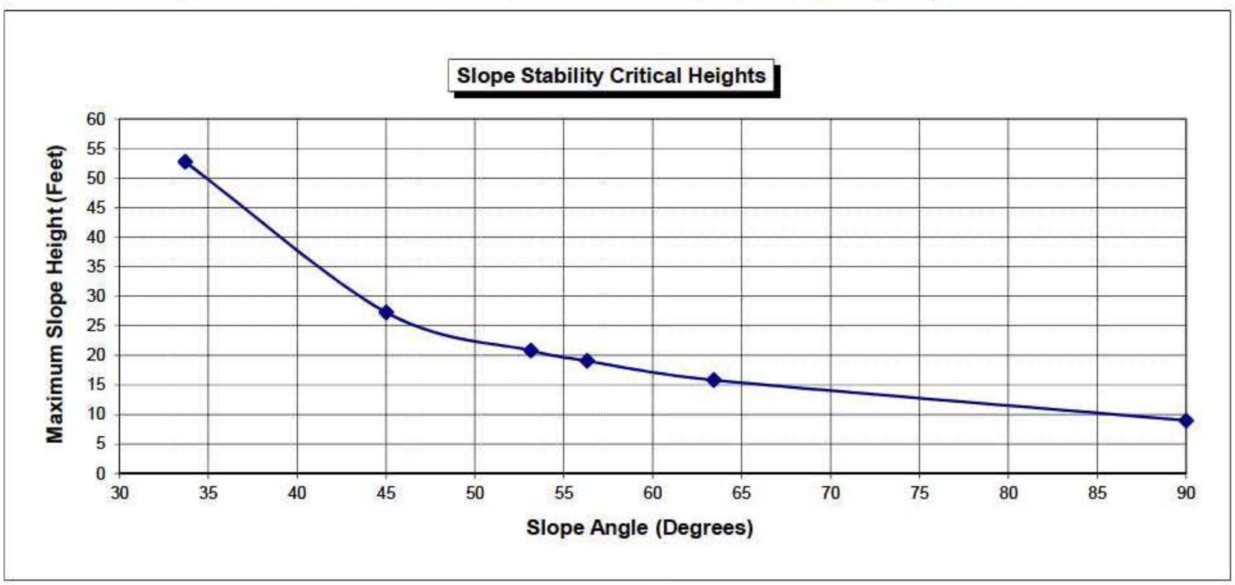
$$(\phi_\delta) = \text{ArcTan}[(\text{Tan}\phi)/FS]$$

$$N = \frac{c}{(\gamma)(H)(FS)}$$

$$H = \frac{c}{(\gamma)(N)(FS)}$$

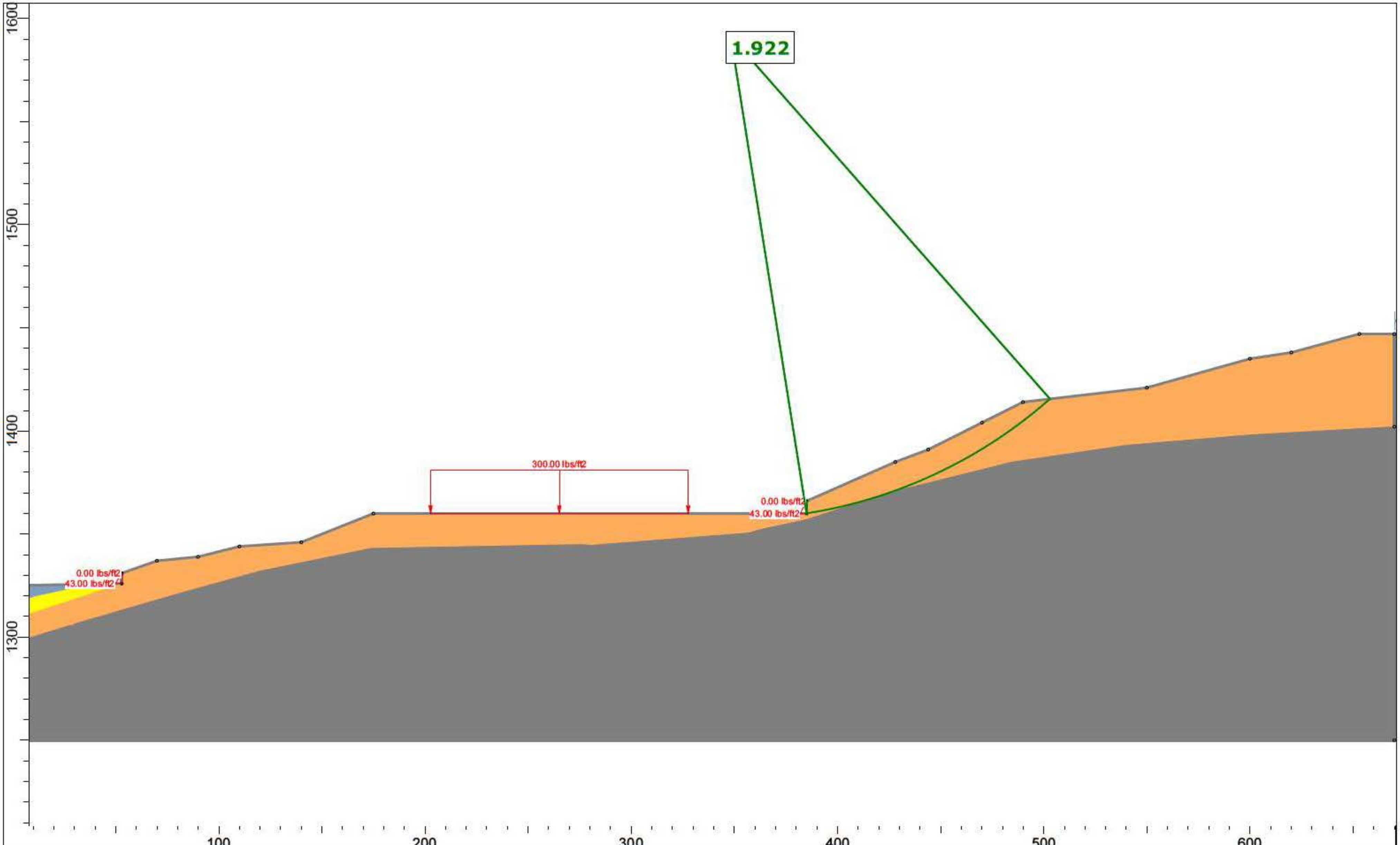
Slope Angle (h:v)	Slope Angle (Degrees)	Maximum Height (Feet)
$1^1/_2 : 1$	33.69	53
1 : 1	45.00	27
$^3/_4 : 1$	53.13	21
$1 : 1^1/_2$	56.30	19
$^1/_2 : 1$	63.43	16
Vertical	90.00	9

Assumptions: Slope is uniform, soils are homogeneous, no water seepage, no surcharge loads.



SLOPE STABILITY ANALYSES RESULTS

(24 PAGES)



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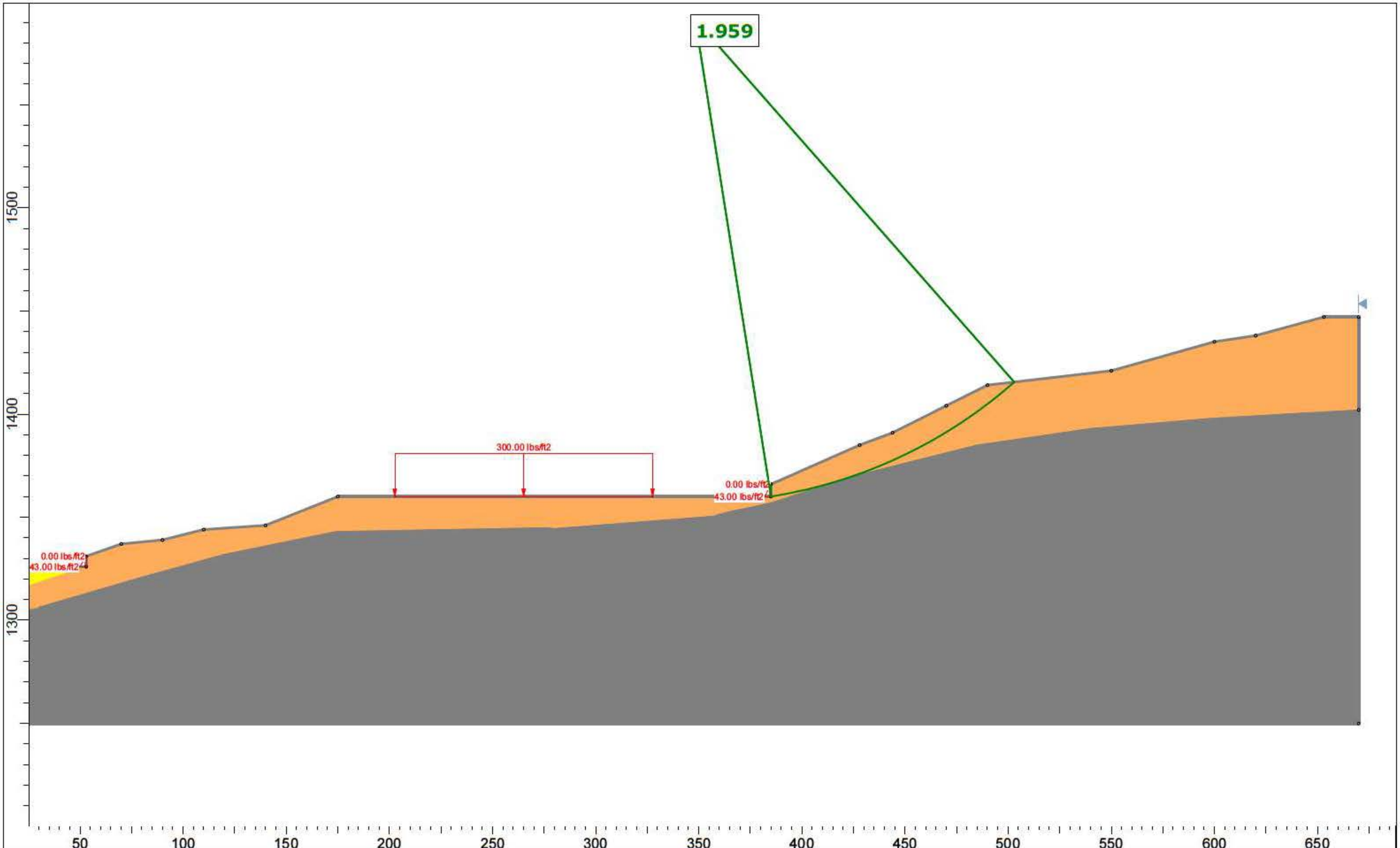

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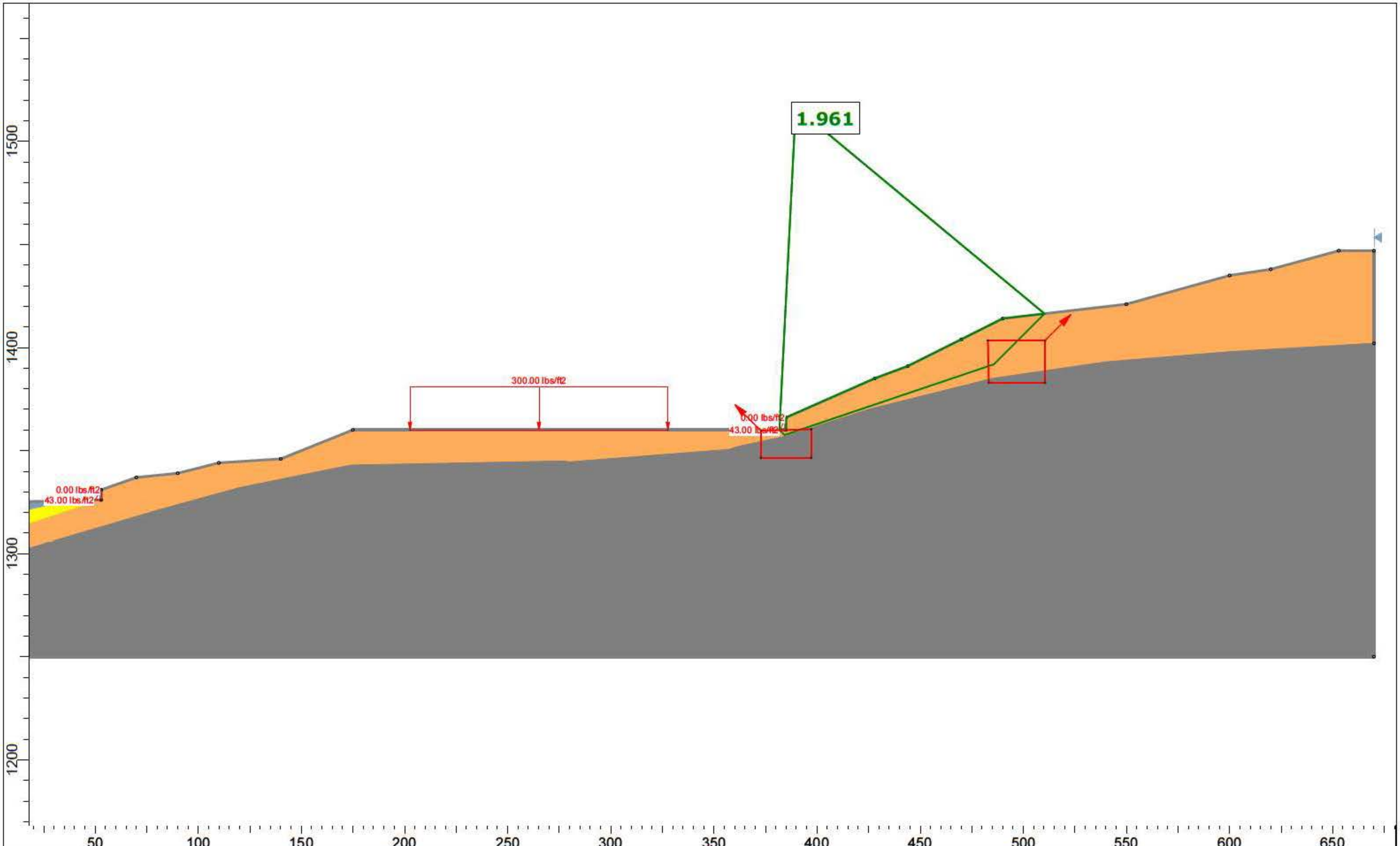

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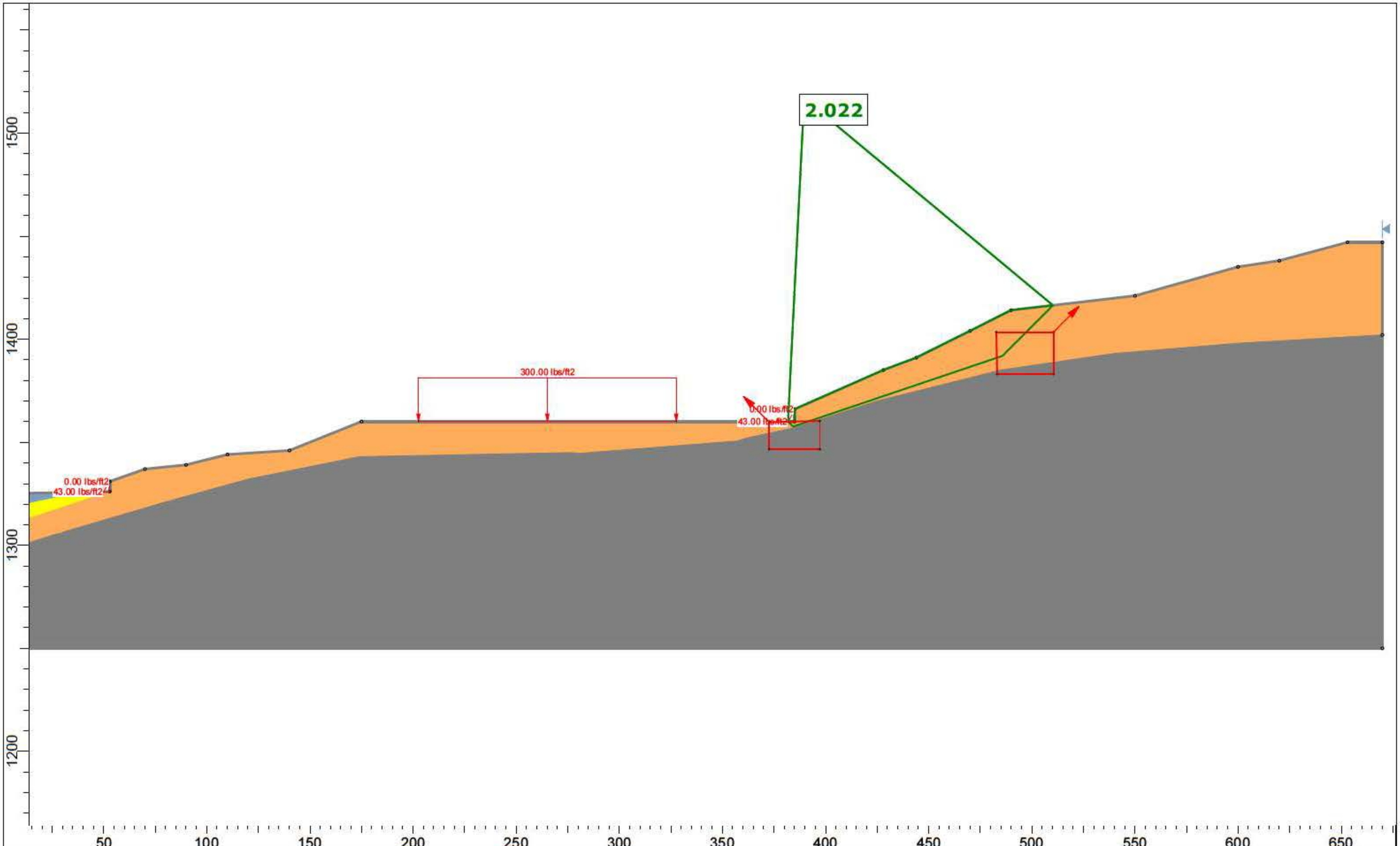

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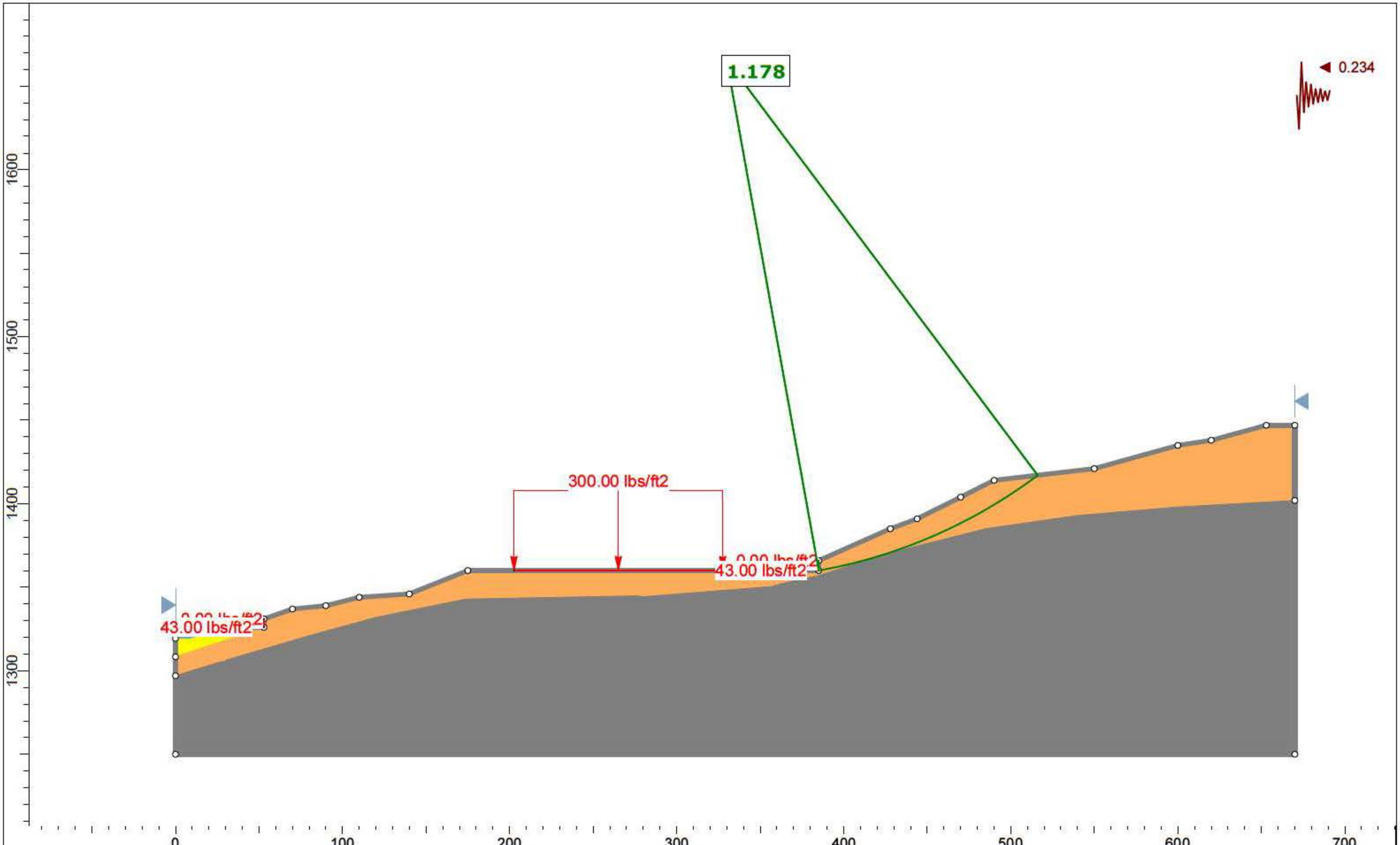

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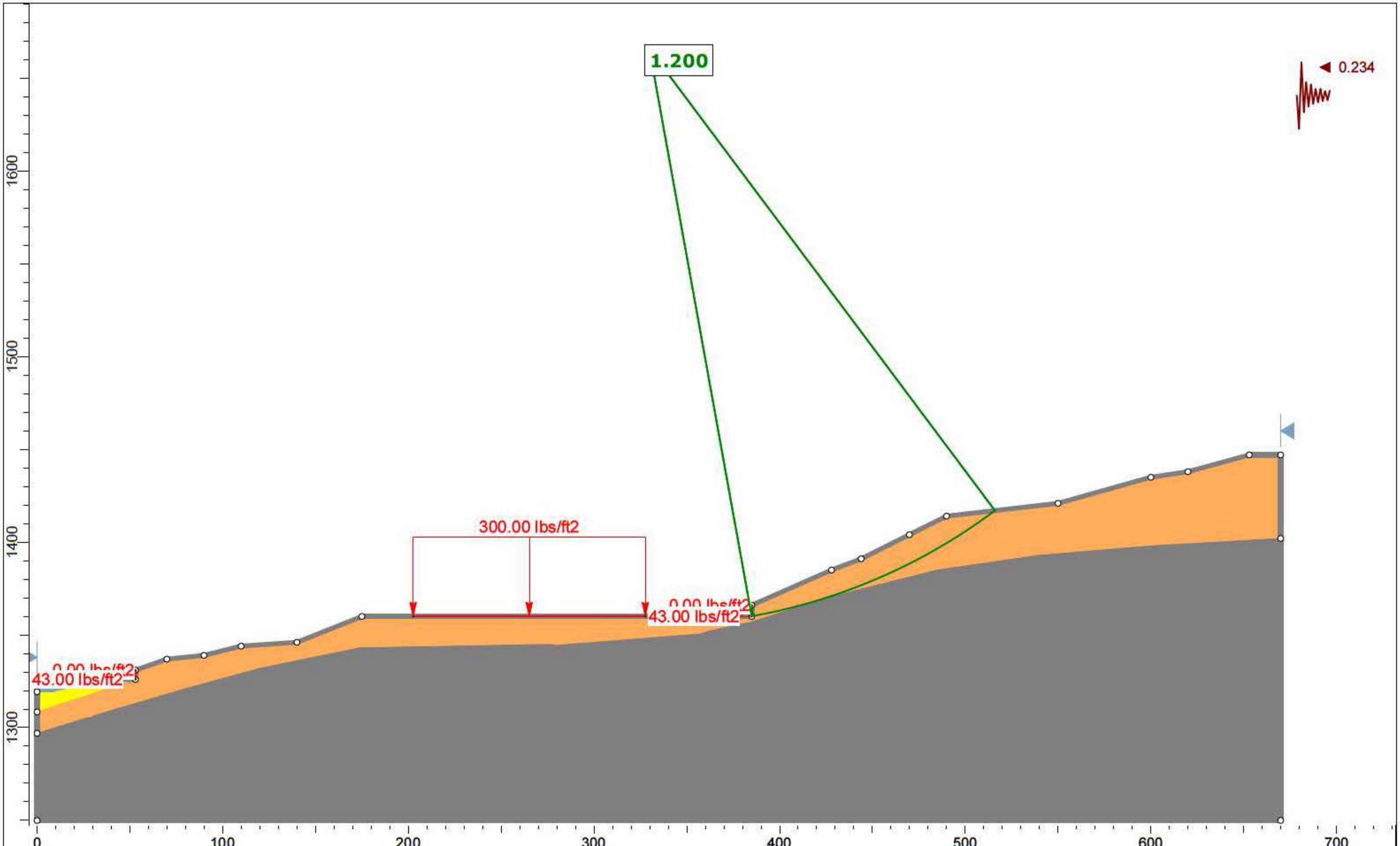

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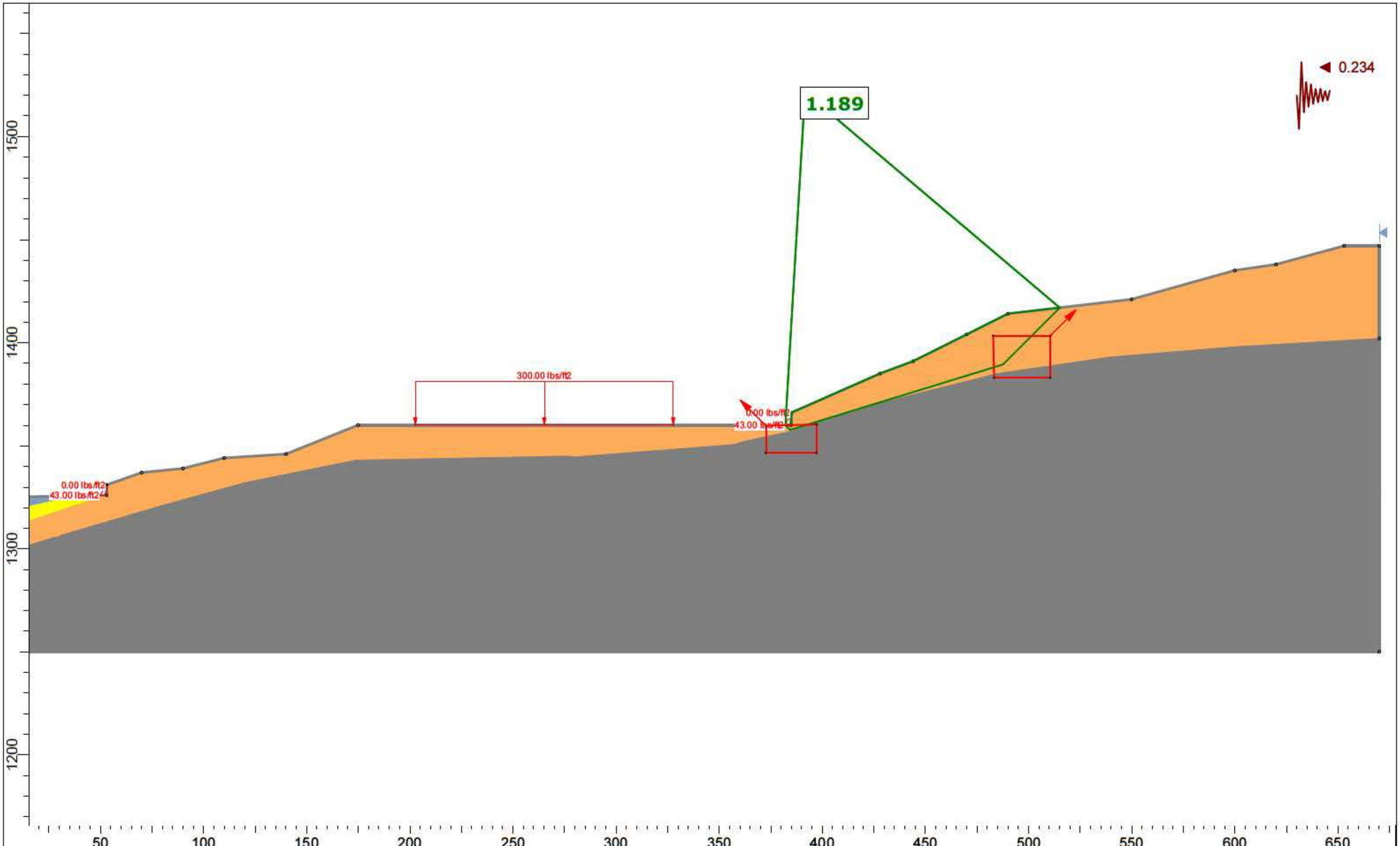

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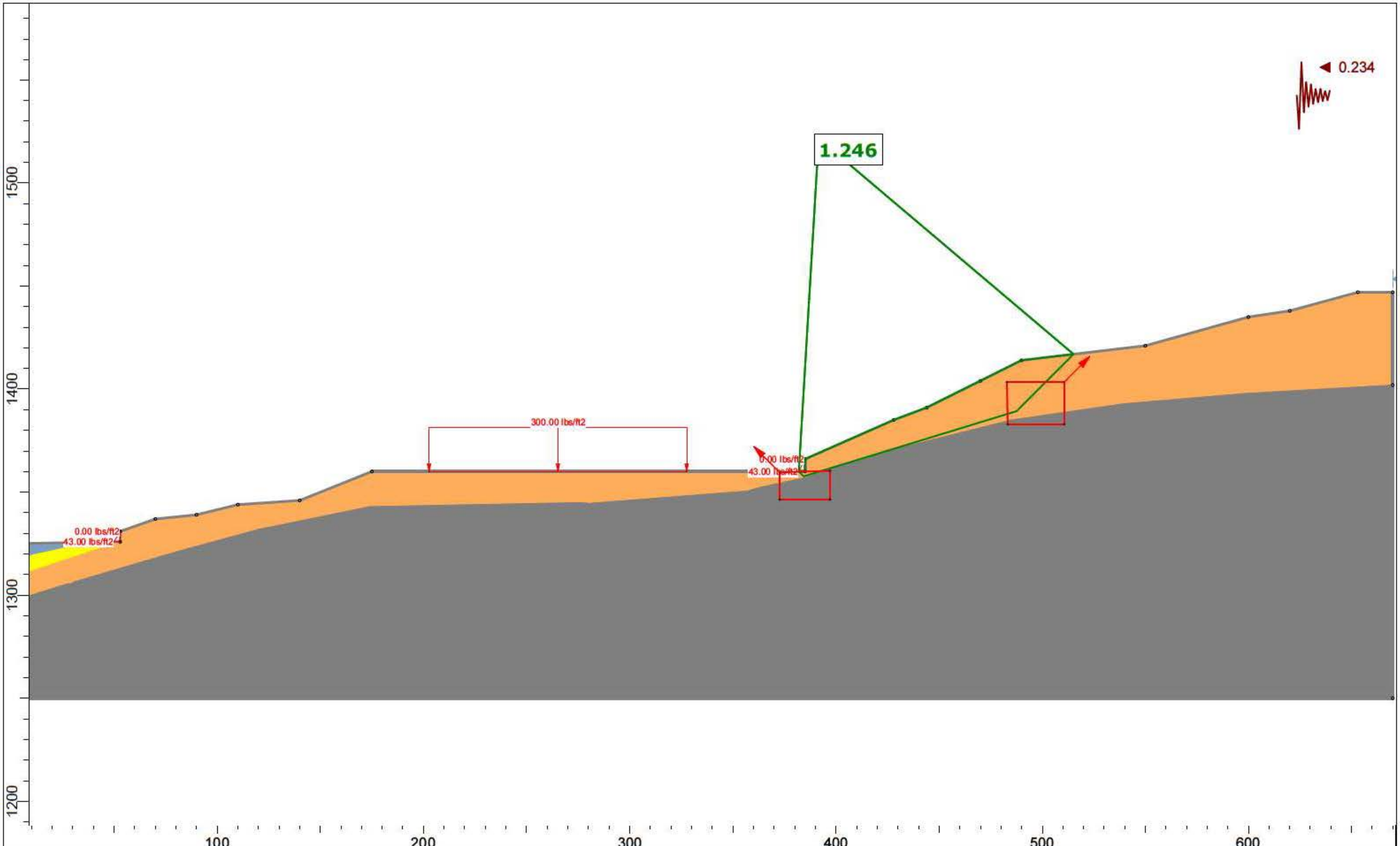

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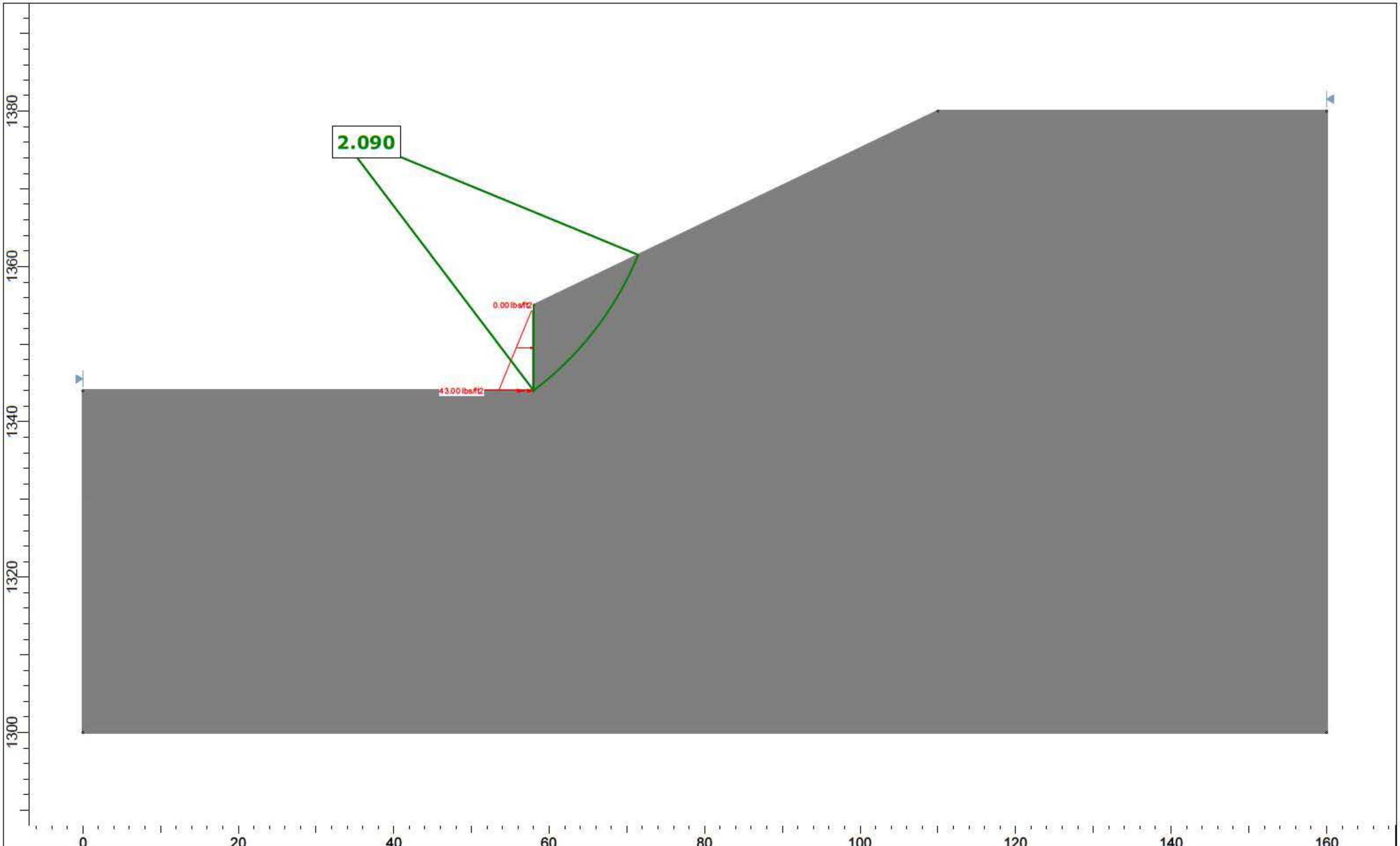

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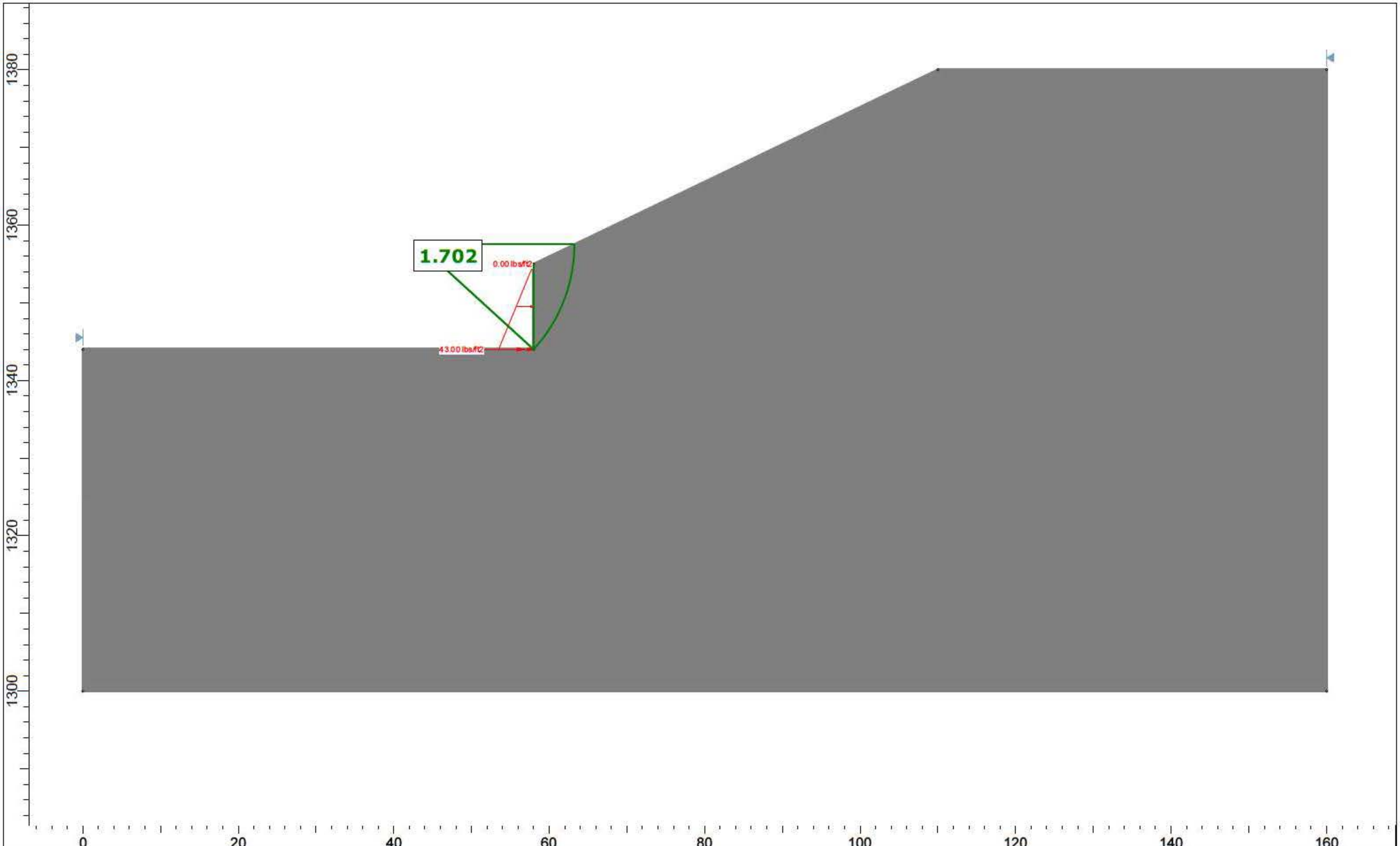

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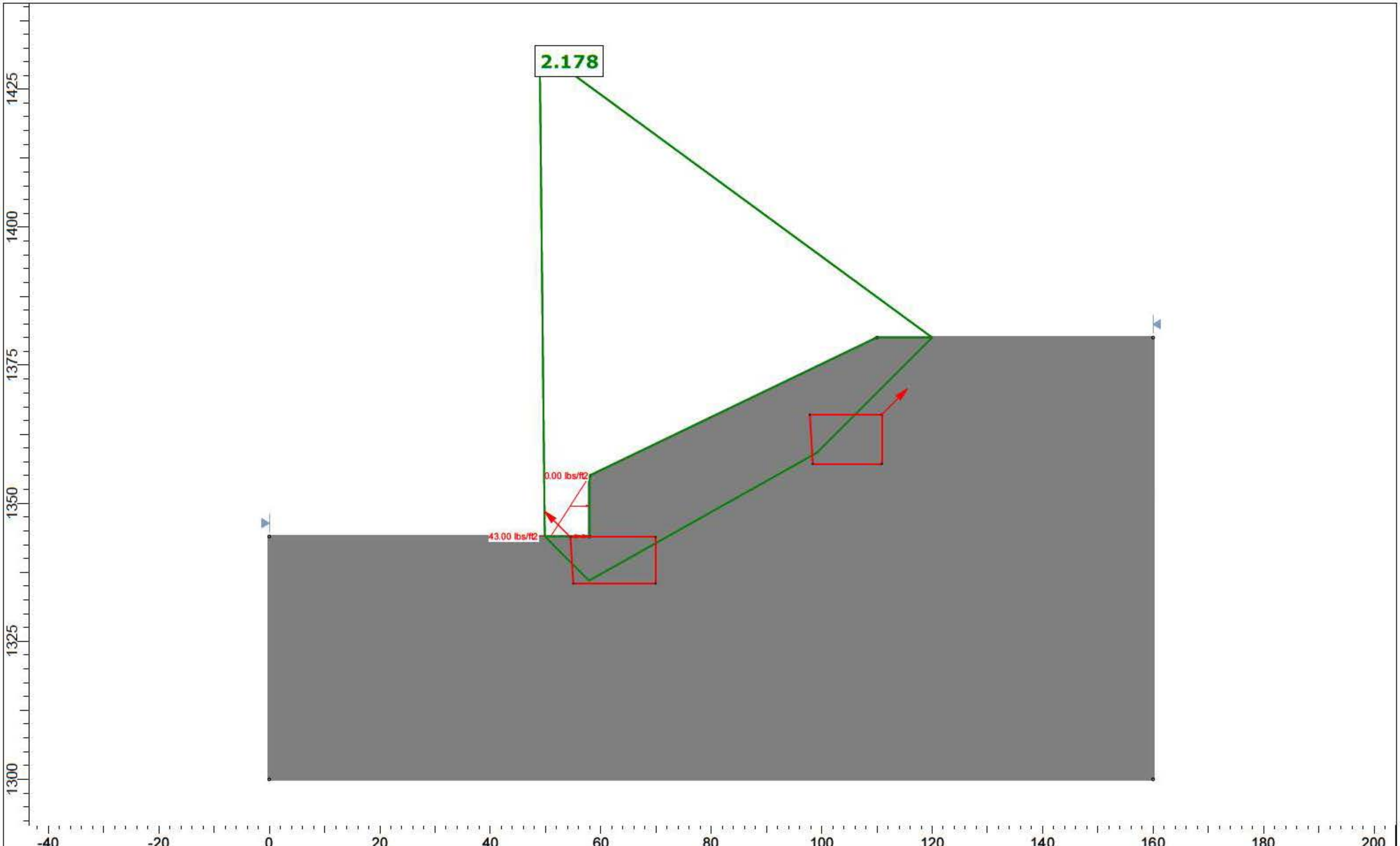

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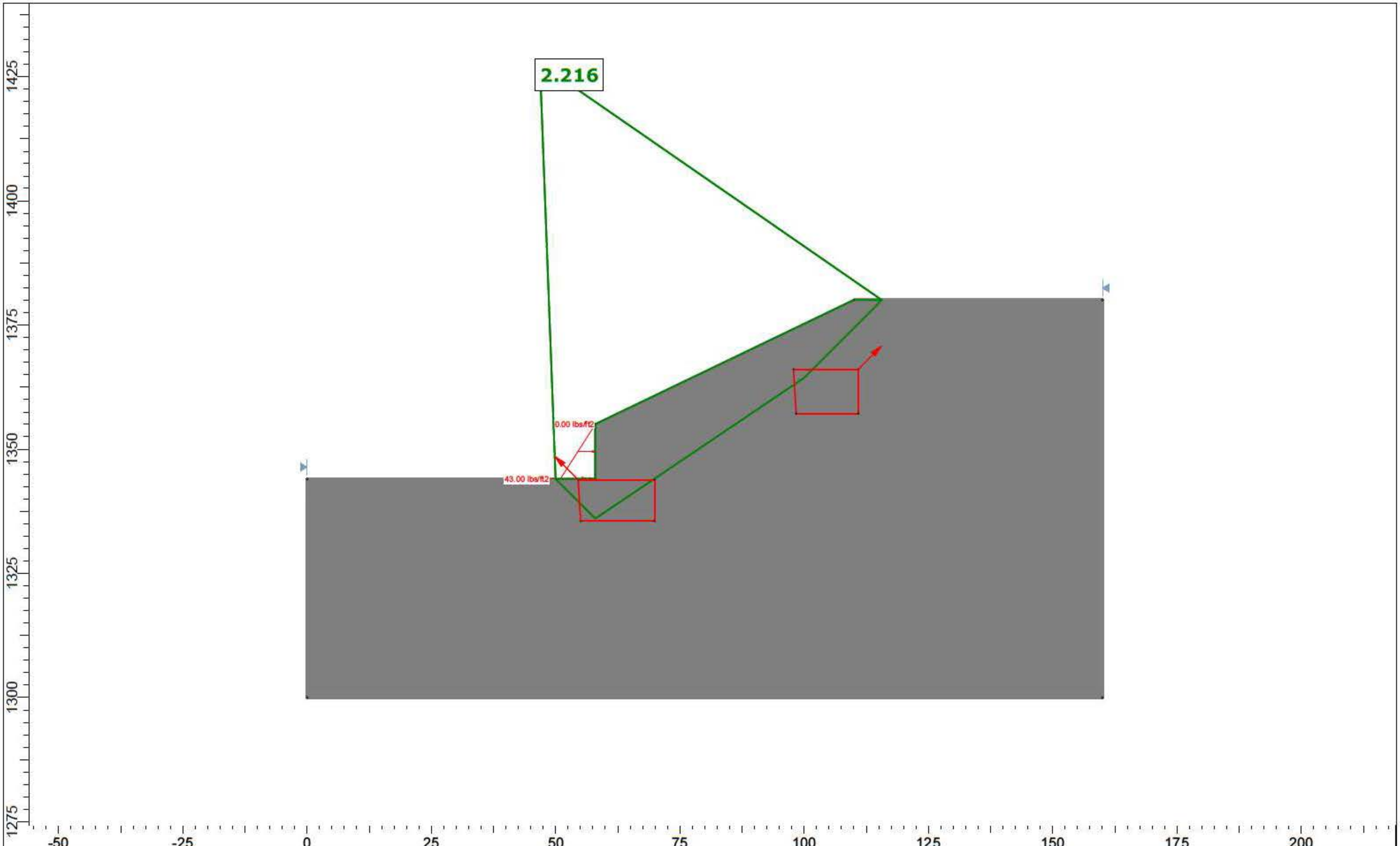

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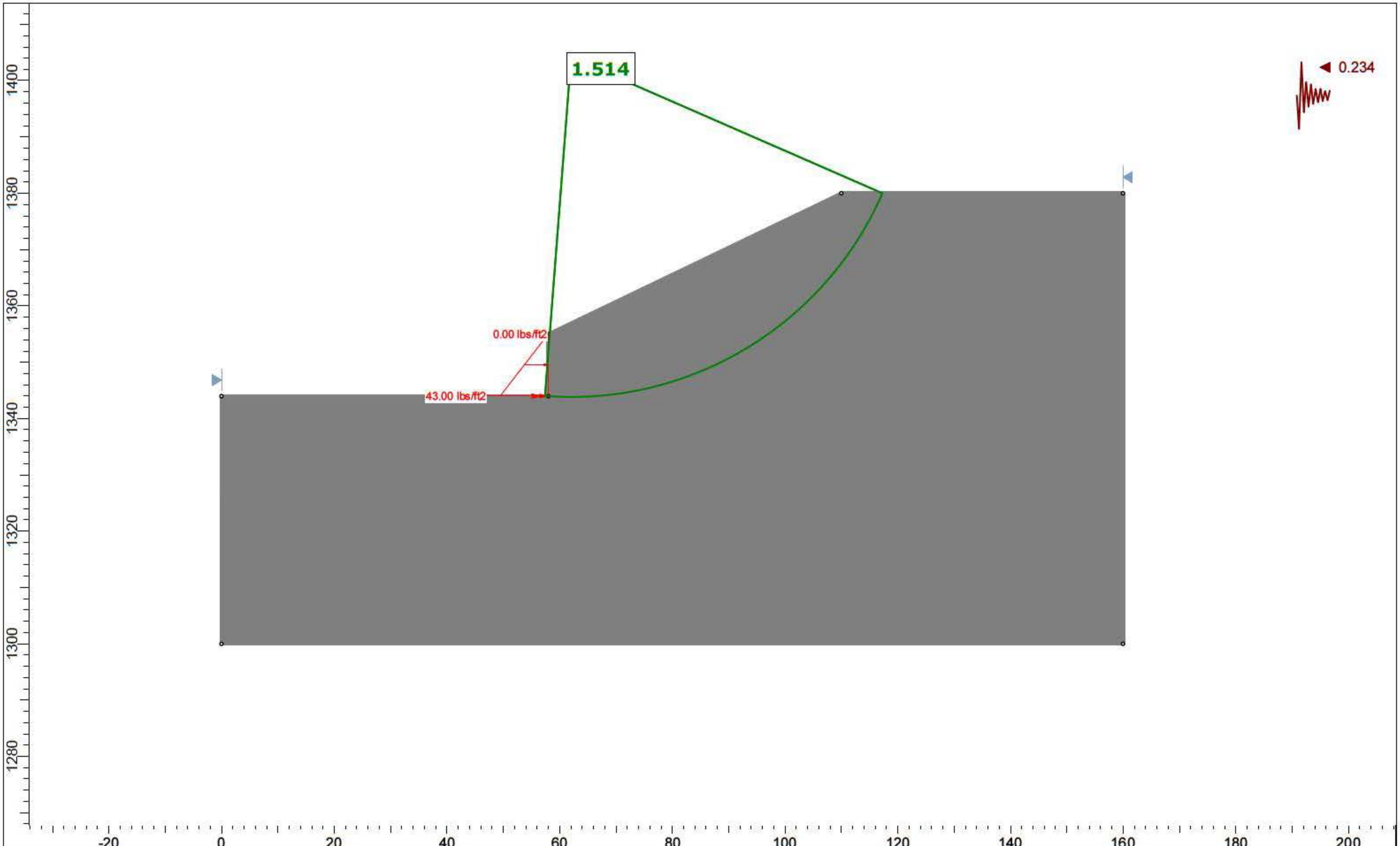

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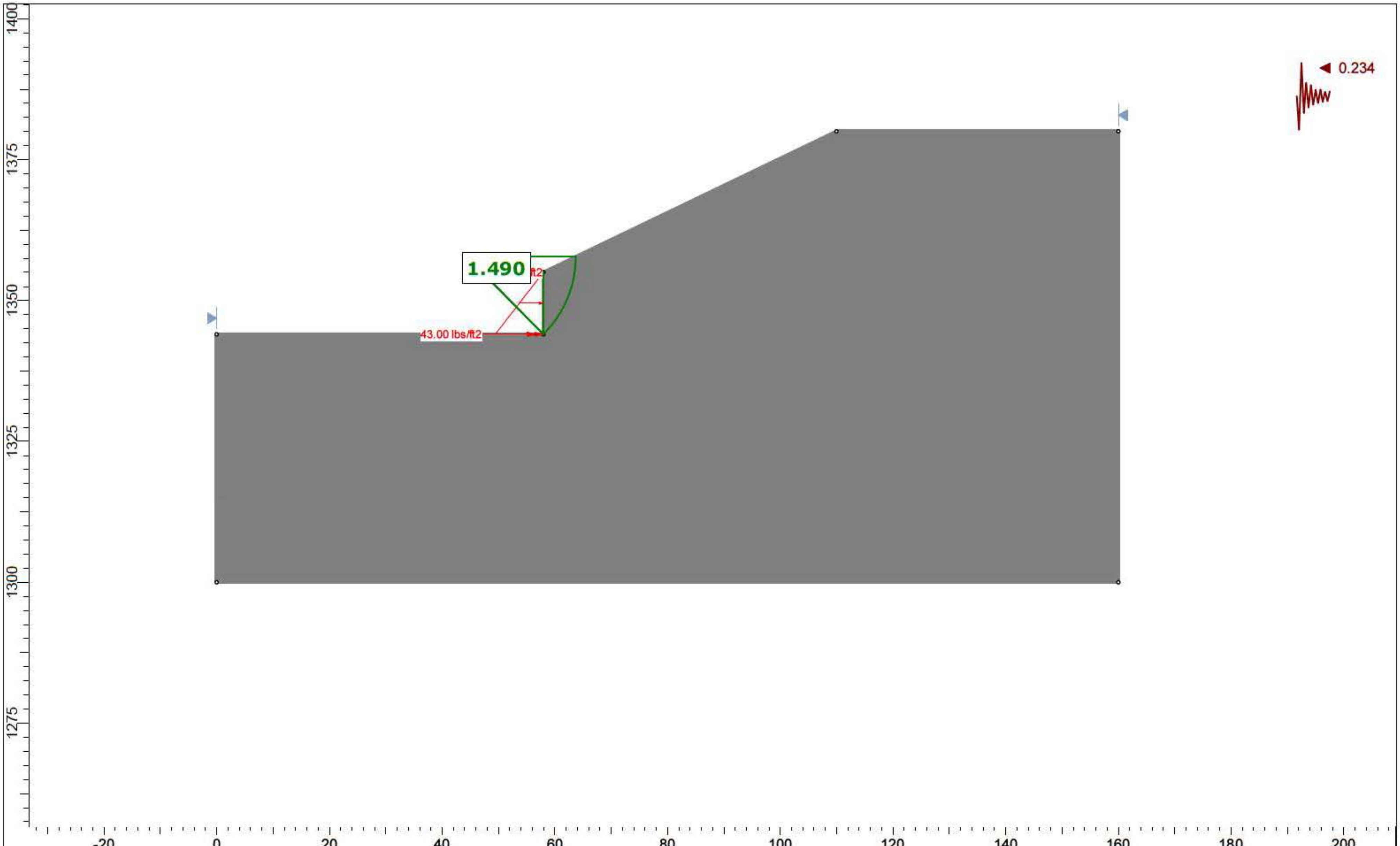

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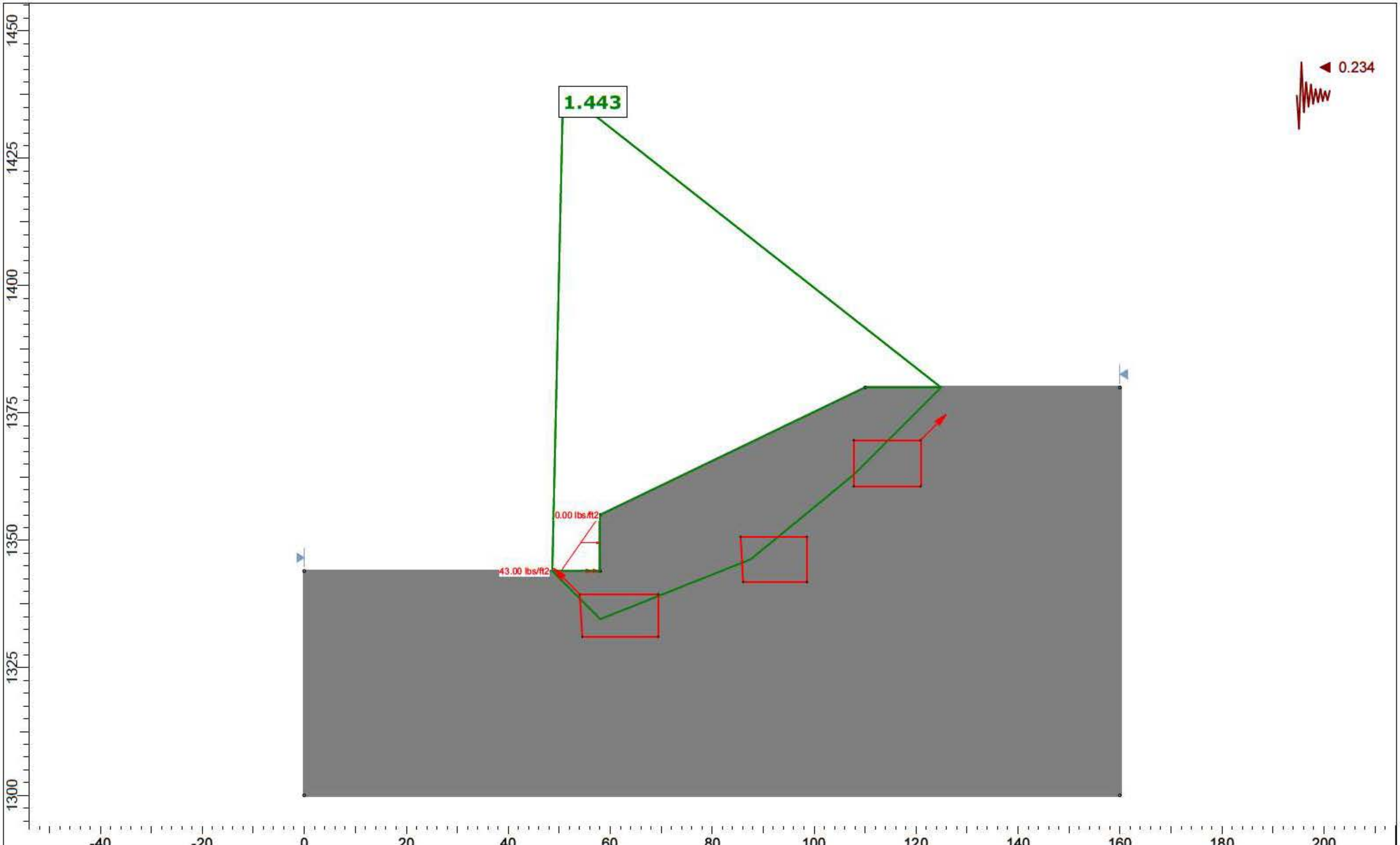

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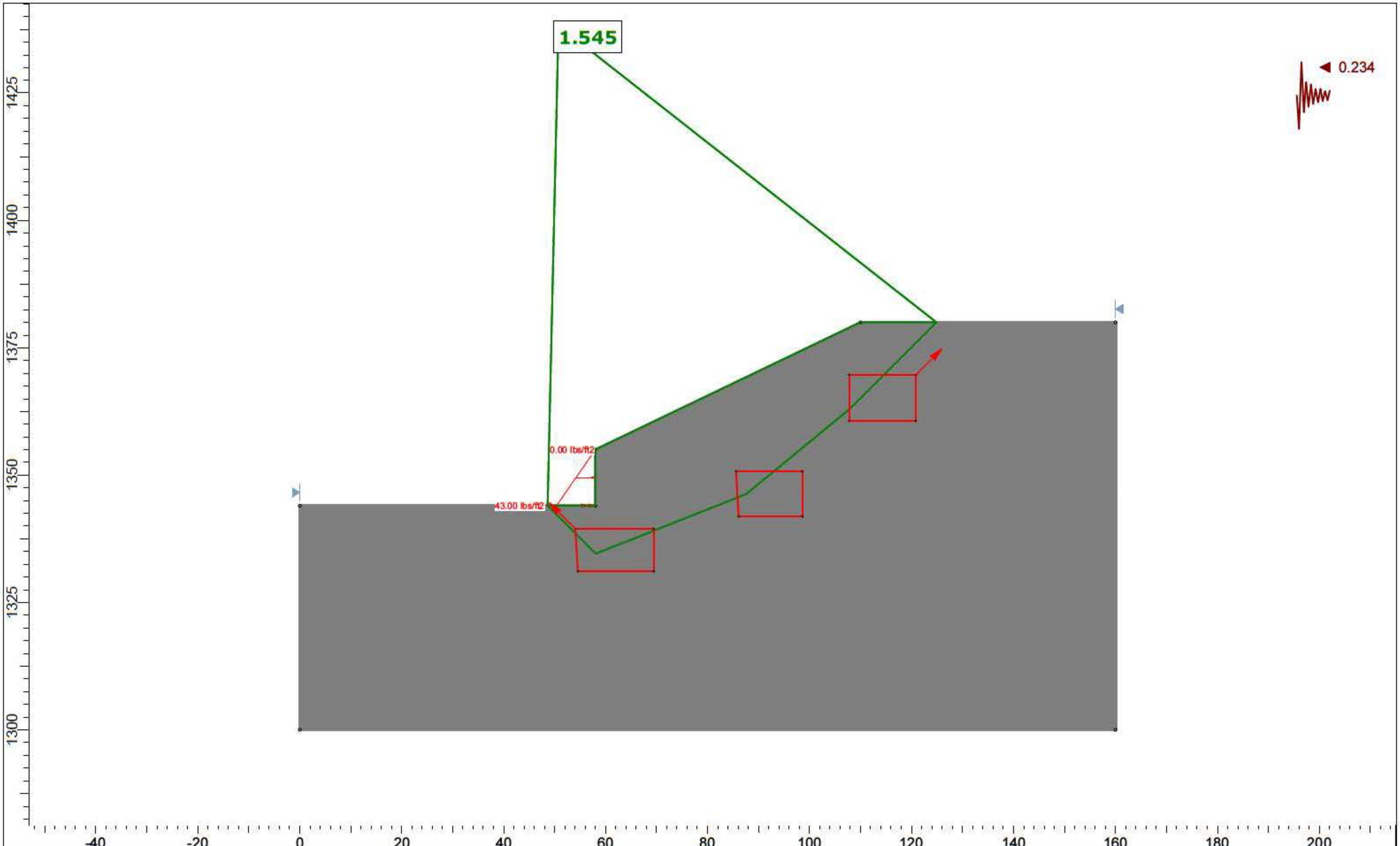

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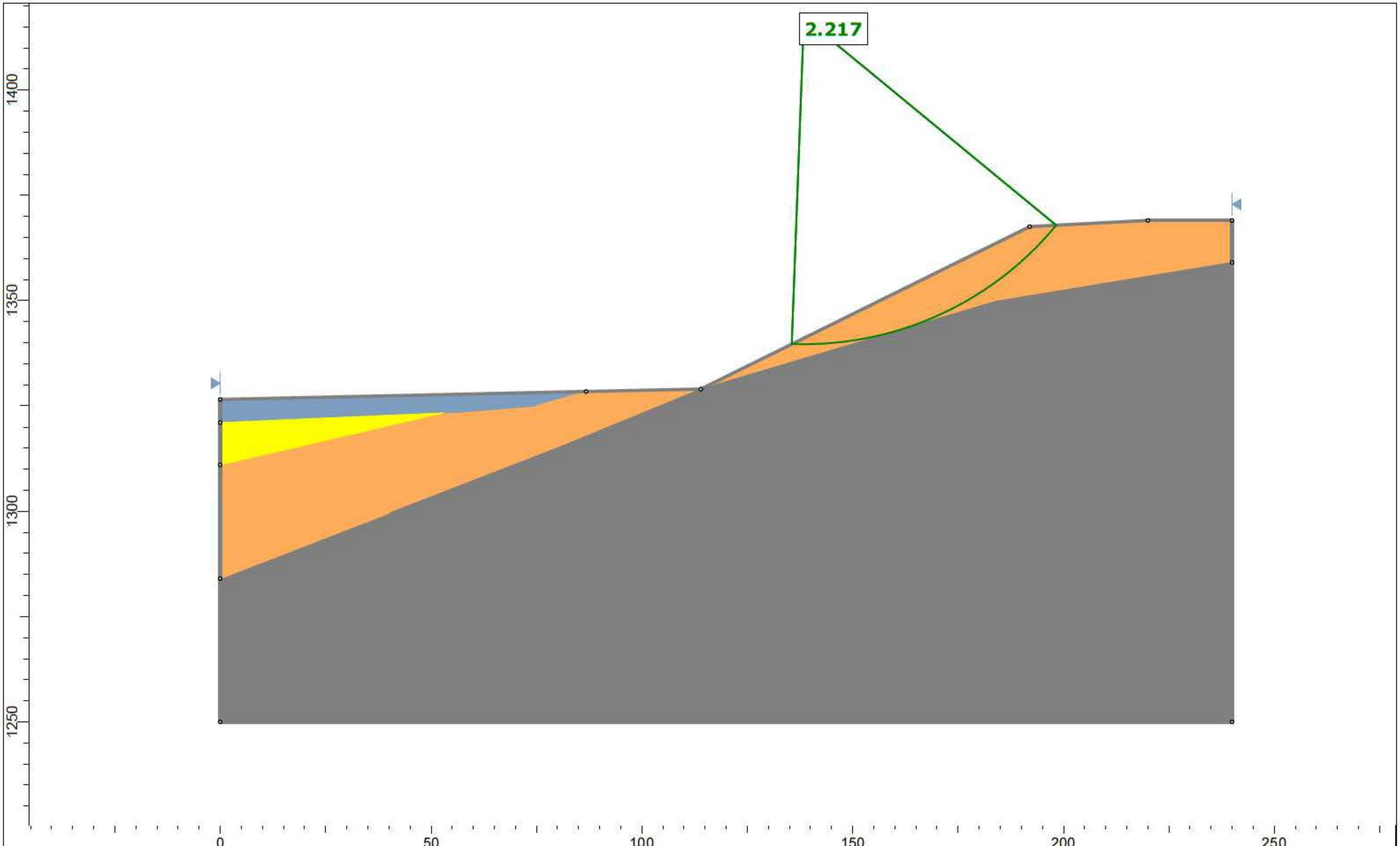

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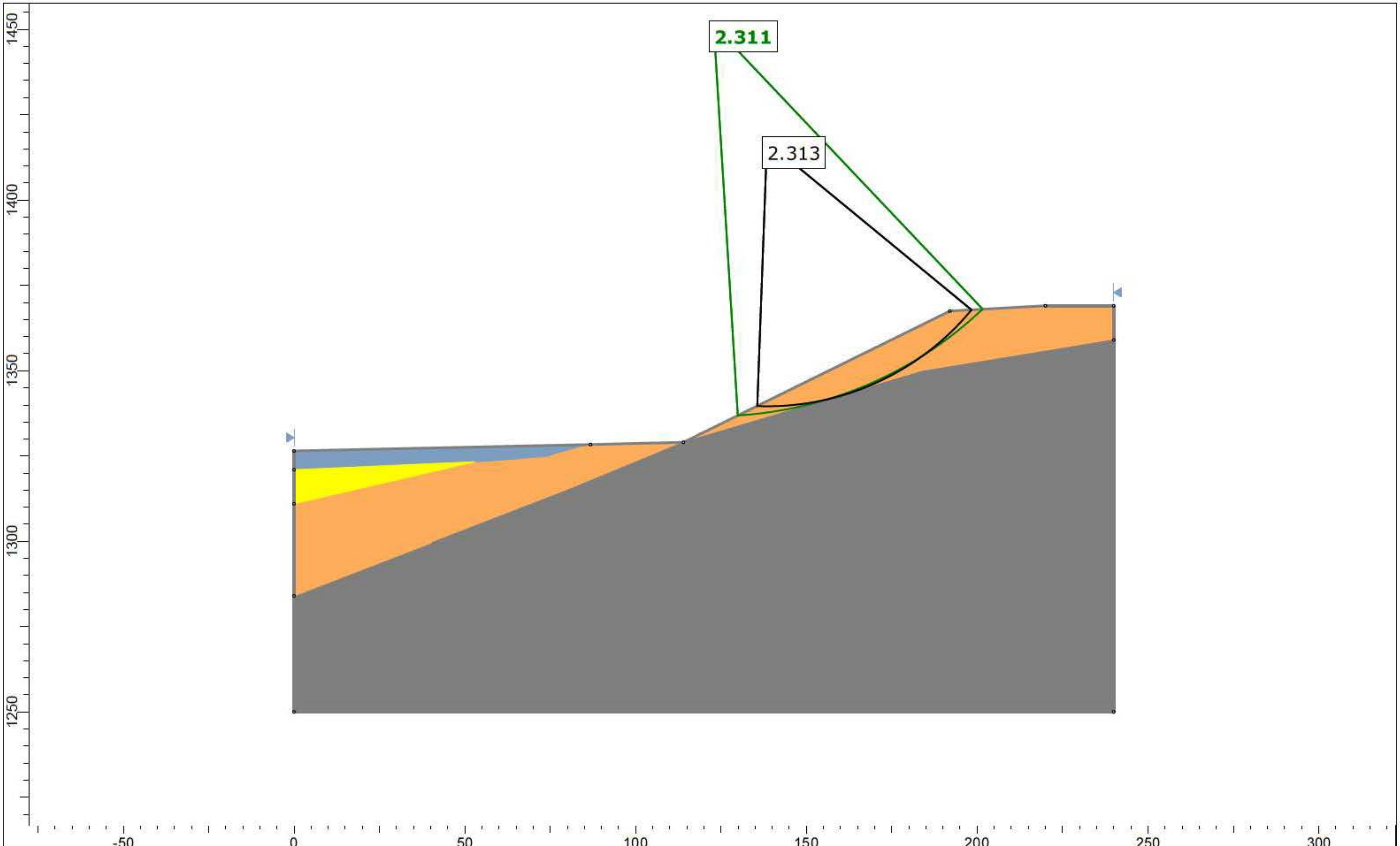

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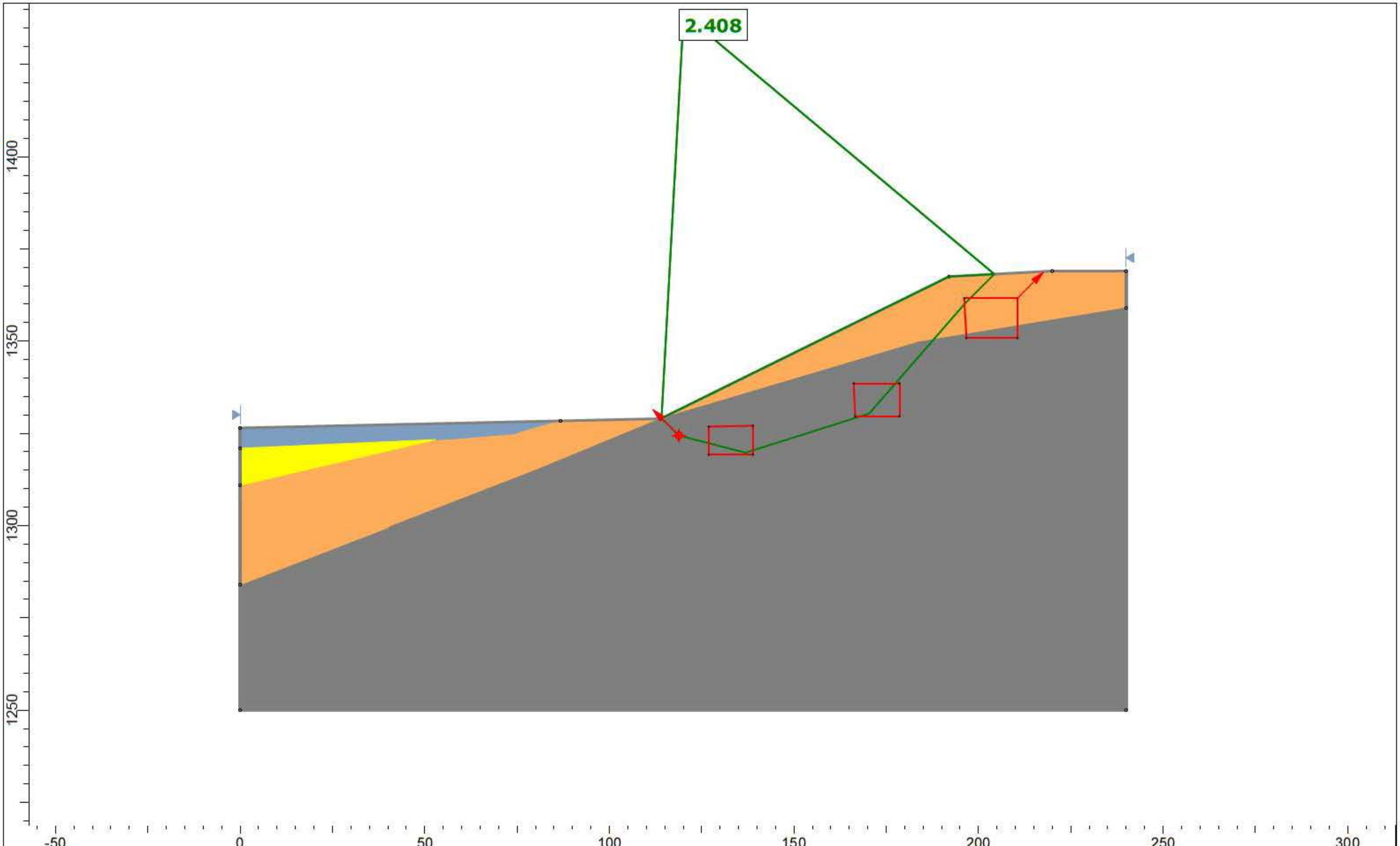

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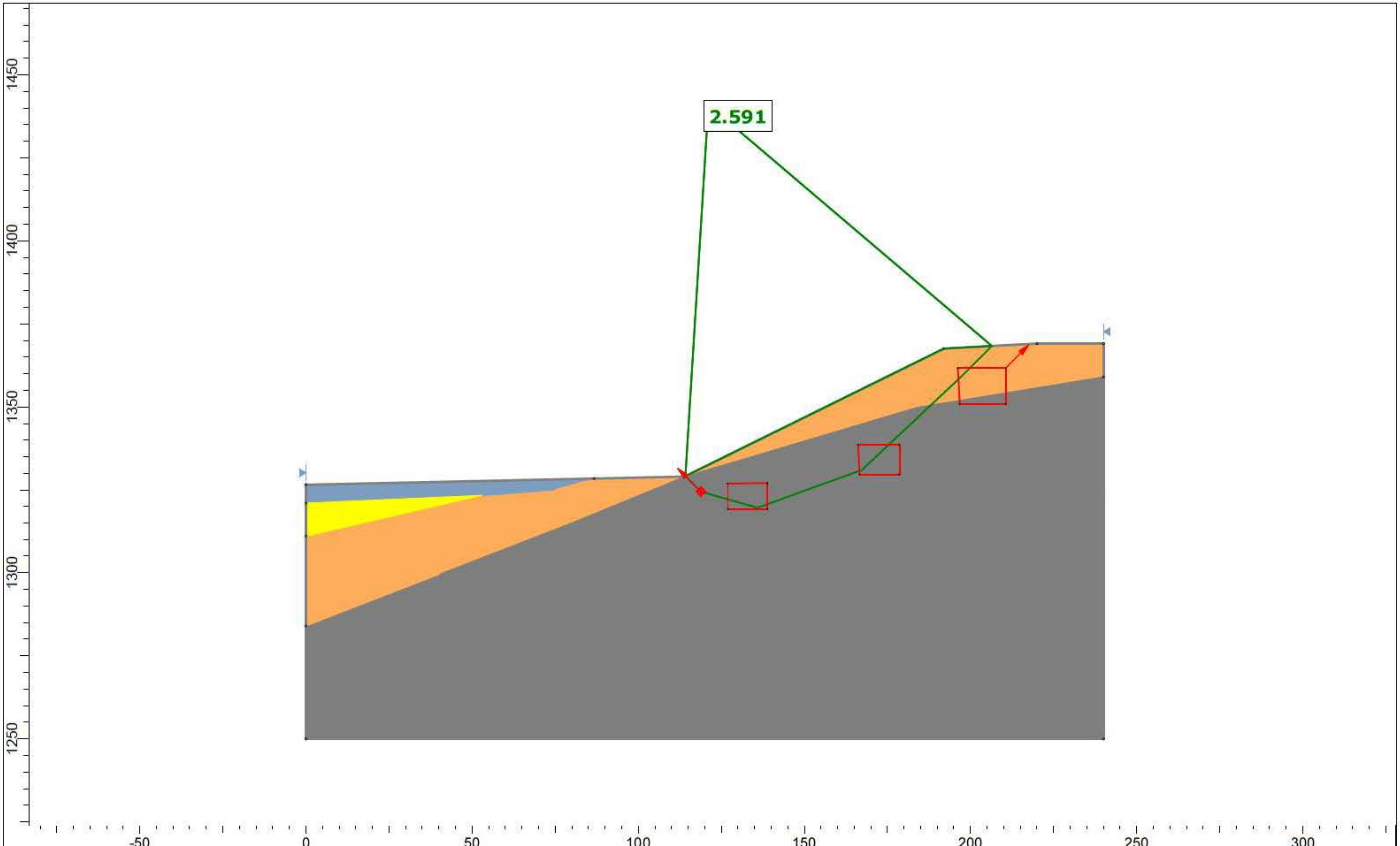

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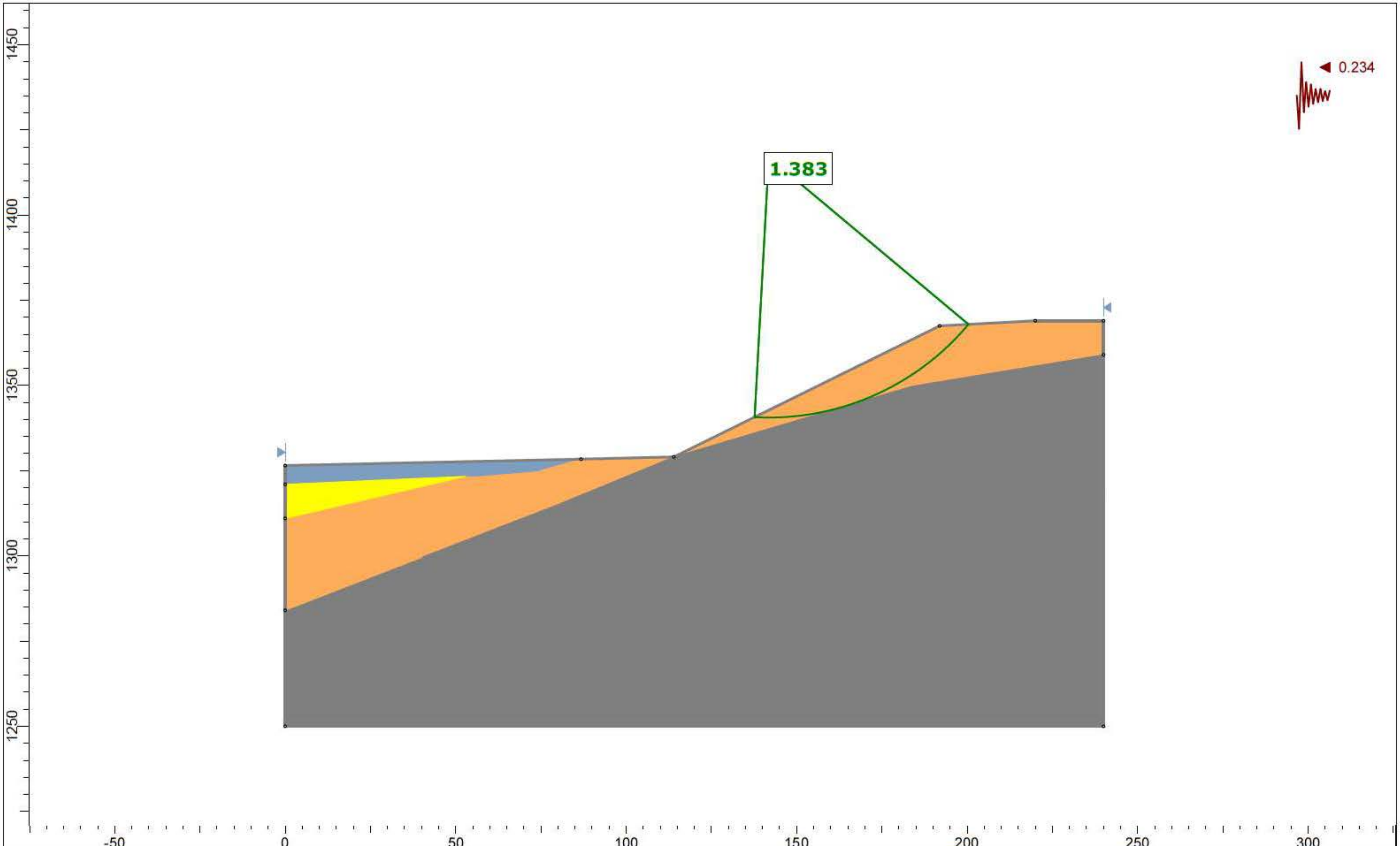

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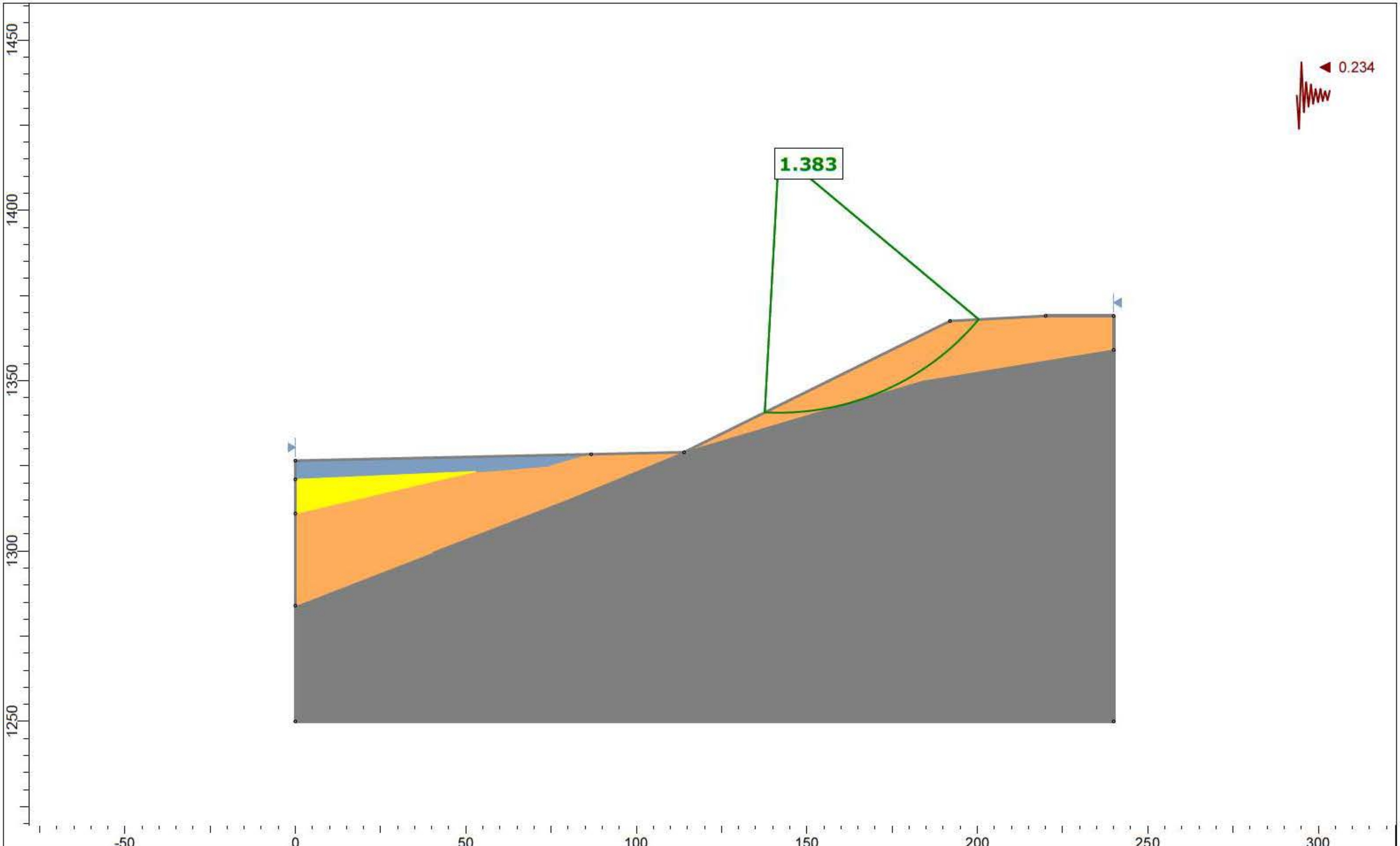

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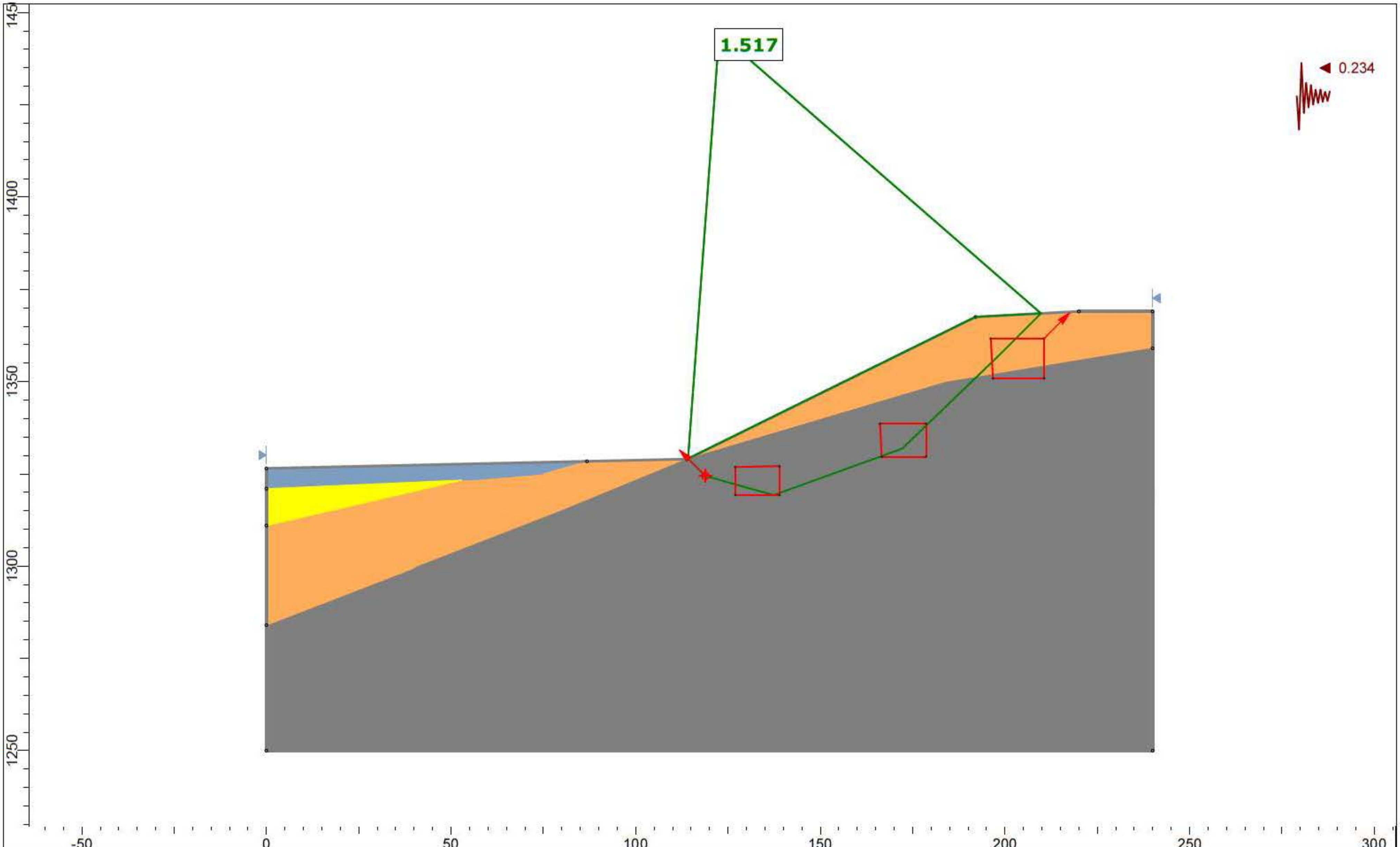

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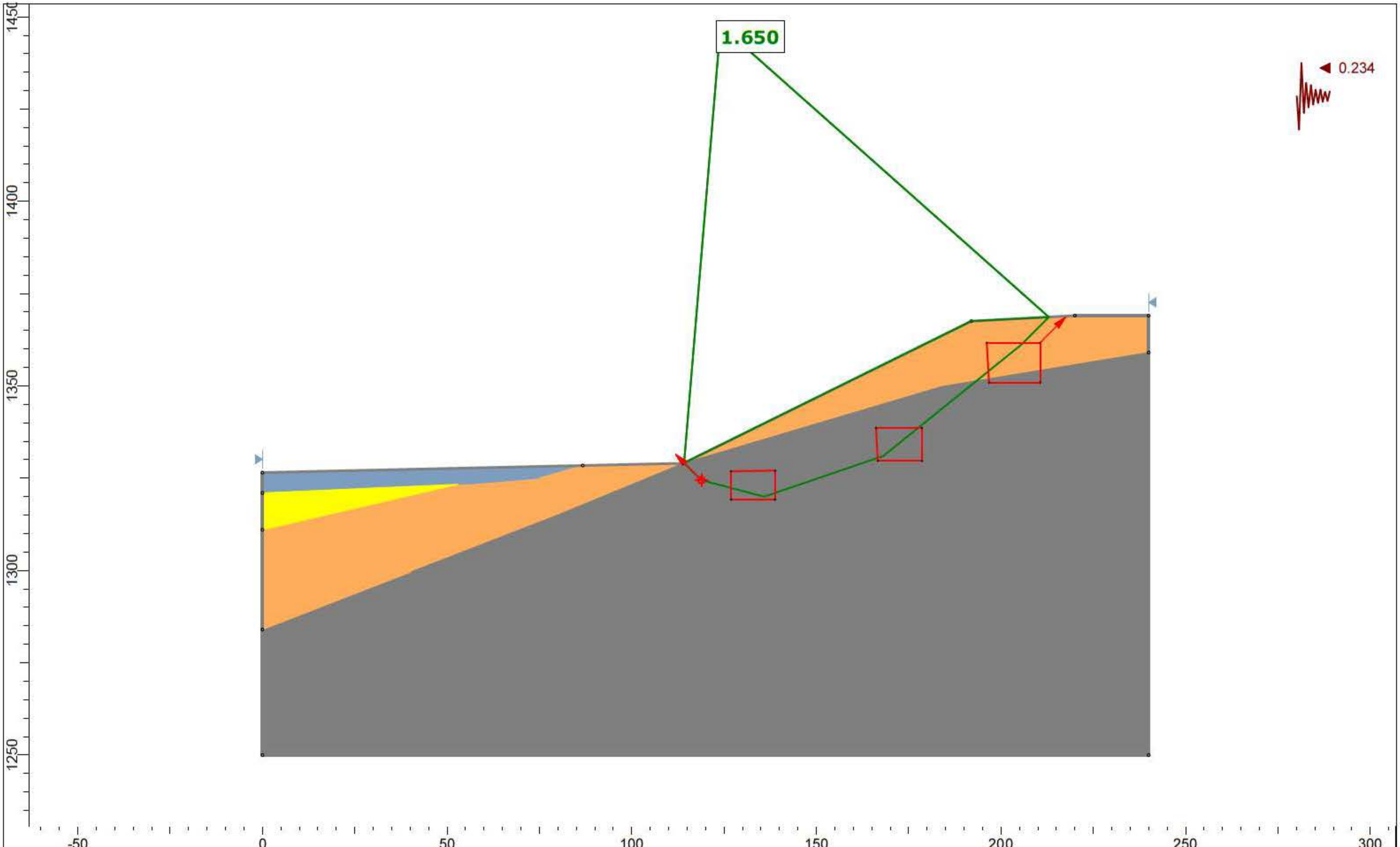

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Date		File Name	22119.DD.static.slmd

Date: 19-Apr-21
File No. 22119
File Name : Chandler Partners

Percolation Rate Calculation for Small Diameter Boring

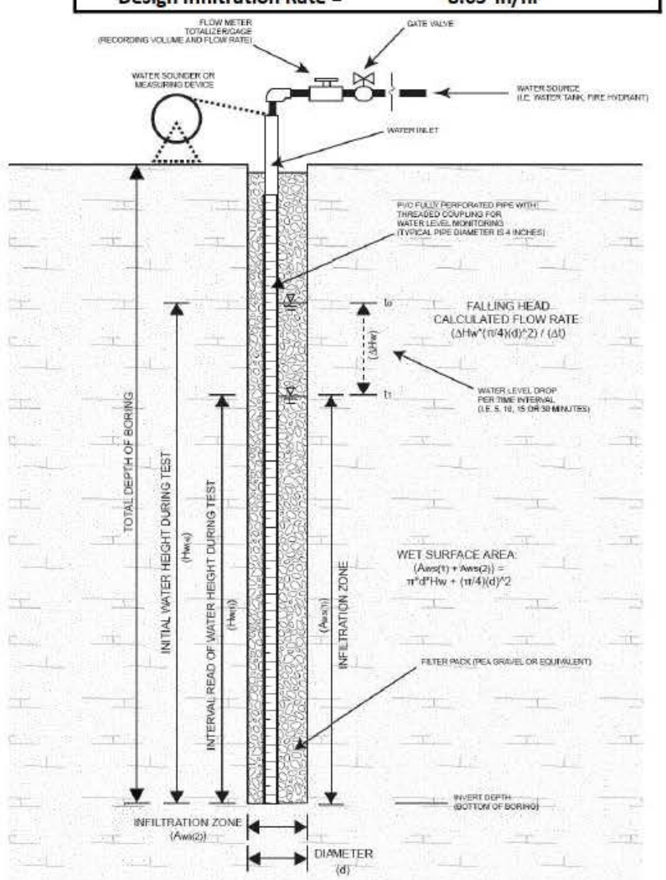
Testing Well Number	1
Boring Diameter (DIA)	8 inches
Depth of Boring	50 feet
Pre-soak Time	2 hours
Measured By	H.C.

Reading Number	Clock Time	Elapsed Time	Water Measurement (d_i) and (d_f)	Water Level Drop	Rate of Drop Variation		Flow Rate	Wet Surface Area	Pre-Adjusted Infiltration Rate
		Min	feet	in	%		in^3/hr	in^2	in/hr
1	11:10		20.00						
	11:40	30	45.60	307.20			30883.2	1377.3	22.4
2	11:45		20.00						
	12:15	30	45.00	300.00	-2.40		30159.4	1558.2	19.4
3	12:20		20.00						
	12:50	30	44.70	296.40	-1.21		29797.4	1648.7	18.1
4	12:58		20.00						
	13:28	30	44.50	294.00	-0.82		29556.2	1709.0	17.3
5	13:50		20.00						
	14:20	30	44.60	295.20	0.41		29676.8	1678.9	17.7
6	14:30		20.00						
	15:00	30	44.50	294.00	-0.41		29556.2	1709.0	17.3
7									
8									

Note: Calculation based on County of Los Angeles, Administrative Manual, Low Impact Development Best Management PracticeGuideline for Design, Investigation, and Reporting, dated 6/30/21.
LA County Minimum 0.3 inches per hour

Raw Percolation Rate= 17.3 in/hr
RF_t= 2
RF_v= 1
RF_s= 1

Design Infiltration Rate = 8.65 in/hr



Date: 19-Apr-21
File No. 22119
File Name : Chandler Partners

Percolation Rate Calculation for Small Diameter Boring

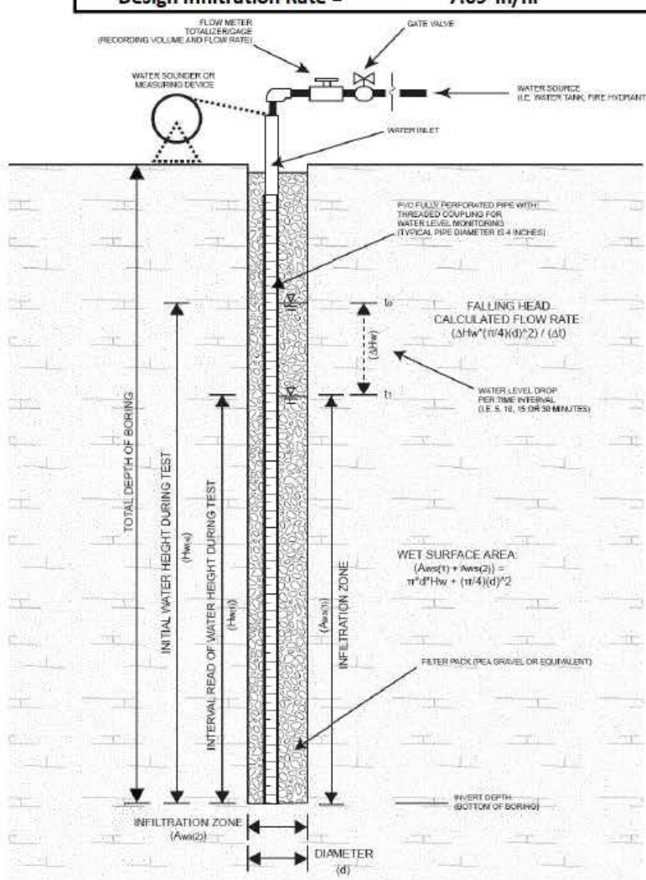
Testing Well Number 2
Boring Diameter (DIA) 8 inches
Depth of Boring 20 feet
Pre-soak Time 2 hours
Measured By H.C.

Reading Number	Clock Time	Elapsed Time	Water Measurement (d_i) and (d_f)	Water Level Drop	Rate of Drop Variation		Flow Rate	Wet Surface Area	Pre-Adjusted Infiltration Rate
		Min	feet	in	%		in^3/hr	in^2	in/hr
1	10:10		10.00						
	10:55	15	17.30	87.60			17613.1	864.6	20.4
2	11:00		10.00						
	11:15	15	17.00	84.00	-4.29		16889.2	955.0	17.7
3	11:20		10.00						
	11:35	15	16.80	81.60	-2.94		16406.7	1015.4	16.2
4	11:39		10.00						
	11:54	15	16.60	79.20	-3.03		15924.1	1075.7	14.8
5	12:02		10.00						
	12:17	15	16.50	78.00	-1.54		15682.9	1105.8	14.2
6	12:23		10.00						
	12:38	15	16.50	78.00	0.00		15682.9	1105.8	14.2
7									
8									

Note: Calculation based on County of Los Angeles, Administrative Manual, Low Impact Development Best Management PracticeGuideline for Design, Investigation, and Reporting, dated 6/30/21.
LA County Minimum 0.3 inches per hour

Raw Percolation Rate= 14.2 in/hr
RF_t= 2
RF_v= 1
RF_s= 1

Design Infiltration Rate = 7.09 in/hr



Soil Corrosivity Evaluation Report for Chandler Partners

February 11, 2022

Prepared for:
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Glendale, CA, 91201
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Project X Job #: S220209E
Client Job or PO #: 22119

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1 Executive Summary

A corrosion evaluation of the soils at Chandler Partners was performed to provide corrosion control recommendations for general construction materials. The site is located at 23755 Newhall Avenue, Santa Clarita, California. Six (6) samples were tested to a depth of 3.0 ft. Site ground water and topography information was provided by Geotechnologies, Inc.. Groundwater depth was determined to be Greater than 50ft feet below finished grade.

Every material has its weakness. Aluminum alloys, galvanized/zinc coatings, and copper alloys do not survive well in very alkaline or very acidic pH environments. Copper and brasses do not survive well in high nitrate or ammonia environments. Steels and irons do not survive well in low soil resistivity and high chloride environments. High chloride environments can even overcome and attack steel encased in normally protective concrete. Concrete does not survive well in high sulfate environments. And nothing survives well in high sulfide and low redox potential environments with corrosive bacteria. This is why Project X tests for these 8 factors to determine a soil's corrosivity towards various construction materials. **Depending solely on soil resistivity or Caltrans corrosion guidelines (which concentrate on concrete/steel highways), will over-simplify descriptions as corrosive or non-corrosive. This approach will not detect these other factors attacking other metals because it is possible to have bad levels of corrosive ions and still have greater than 1,100 ohm-cm soil resistivity. We have observed this fact on thousands of soil samples tested in our laboratory.**

It should not be forgotten that import soil should also be tested for all factors to avoid making your site more corrosive than it was to begin with.

The recommendations outlined herein are not a substitute for any design documents previously prepared for the purpose of construction and apply only to the depth of samples collected.

Soil samples were tested for minimum resistivity, pH, chlorides, sulfates, ammonia, nitrates, sulfides and redox.

As-Received soil resistivities ranged between 9,380 ohm-cm and 542,700.0 ohm-cm. This data would be similar to a Wenner 4 pin test in the field and used in the design of a cathodic protection or grounding bed system. This resistivity can change seasonally depending on the weather and moisture in the ground. This reading alone can be misleading because condensation or minor water leaks will occur underground along pipe surfaces creating a saturated soil environment in the trench on infrastructure surfaces. This is why minimum or saturated soil resistivity measurements are more important than as-received resistivities.

Saturated soil resistivities ranged between 1,072 ohm-cm to 8,040 ohm-cm.

The worst of these values is considered to be corrosive to general metals.

PH levels ranged between 6.3 to 7.6 pH. PH levels were determined to be at levels not detrimental to copper or aluminum alloys. The pH of these samples can allow corrosion of steel and iron in moist environments.

Chlorides ranged between 4 mg/kg to 36 mg/kg. Chloride levels in these samples are low and may cause insignificant corrosion of metals.

Sulfates ranged between 41 mg/kg to 326 mg/kg. Sulfate levels in these samples are negligible for corrosion of cement. Any type of cement can be used that does not contain encased metal.

Ammonia ranged between 6.4 mg/kg to 43.2 mg/kg. Nitrates ranged between 2.9 mg/kg to 41.6 mg/kg. Concentrations of these elements were high enough to cause accelerated corrosion of copper and copper alloys such as brass.

Sulfides presence was determined to be negative. REDOX ranged between + 130 mV to + 169 mV. The probability of corrosive bacteria was determined to be low due to the sulfide and positive REDOX levels determined in these samples.

2 Corrosion Control Recommendations

The following recommendations are based upon the results of soil testing.

2.1 Cement

The highest reading for sulfates was 326 mg/kg or 0.0326 percent by weight.

Per ACI 318-14, Table 19.3.1.1, sulfate levels in these samples categorized as S0 and are negligible for corrosion of metals and cement. Per ACI 318-14 Table 19.3.2.1 any type of cement not containing steel or other metal can be used.

2.2 Steel Reinforced Cement/ Cement Mortar Lined & Coated (CML&C)

Chlorides in soil can overcome the corrosion inhibiting property of cement for steel, as it can also break through passivated surfaces of aluminum and stainless steels.[1,2] The highest concentration of chlorides was 36 mg/kg.

Chloride levels in these samples are not significantly corrosive to metals not in tension. Standard cement cover may be used in these soils.

Though soils at some locations are significantly corrosive to various metals, per ACI 318-14 Chapter 19 Table 19.3.1.1, all slabs on this site exposure categories and class for **Corrosion Protection of Reinforcement (C) would be considered C1** as Concrete exposed to moisture [mud/rain] (slab sides and bottom) but not to an external source of chlorides. Though there are chlorides in the soil, ACI 318's definition of "external source of chlorides" consists of deicing chemicals, salt, brackish water, seawater, or spray from these sources. The chloride levels in seawater are typically over 19,000 mg/L or 19,000 ppm.

When concrete is tested for water-soluble chloride ion content, the tests should be made at an age of 28 to 42 days. The limits in Per ACI 318-14 Table 5.3.2.1 are to be applied to chlorides contributed from the concrete ingredients, not those from the environment surrounding the concrete.[3]

[1] Design Manual 303: Cement Cylinder Pipe. Ameron. p.65
[2] *Chapter 19, Table 1904.2.2(1), 2012 International Building Code*

[3] ACI 381-14., BUILDING CODE REQUIREMENTS FOR STRUCTURAL CONCRETE (ACI 318-14) AND COMMENTARY (ACI 318R-14)

2.3 Stainless Steel Pipe/Conduit/Fittings

Stainless steels derive their corrosion resistance from their chromium content and oxide layer which needs oxygen to regenerate if damaged. Thus stainless steel is not good for deep soil applications where oxygen levels are extremely low. Stainless steels should not be installed deeper than a plant root zone. Stainless steels typically have the same nobility as copper on the galvanic series and can be connected to copper. If stainless steel must be used, it must be backfilled with soil having greater than 10,000 ohm-cm resistivity and excellent drainage. 304 Stainless steel will also corrode if in contact with carbon materials such as activated carbon. Stainless steel welds should be pickled.

The soil at this site has low probability for anaerobic corrosive bacteria and low chloride levels. Per Nickel Institute guidelines, 304 or 316 Stainless steels can be used in these soils.

2.4 Steel Post Tensioning Systems

The proper sealing of stressing holes is of utmost importance in PT Systems. Cut off excess strand 1/2" to 3/4" back in the hole. Coat or paint exposed anchorage, grippers, and stub of strands with "Rust-o-leum" or equal. After tendons have been coated, the cement contractor shall dry pack blockouts within ten (10) days. A non-shrink, non-metallic, non-porous moisture-insensitive grout (Master EMACO S 488 or equivalent), or epoxy grout shall be used for this purpose. If an encapsulated post-tension system is used, regular non-shrink grout can be used.

Due to the low chloride concentrations measured on samples obtained from this site, post-tensioned slabs should be protected in accordance with soil considered normal (non-corrosive).[4,5] Addition of grease caps to the cut strand at live end anchors can deter construction defect accusations but are not needed.

2.5 Steel Piles

Steel piles are most susceptible to corrosion in disturbed soil where oxygen is available. Further, a dissimilar environment corrosion cell would exist between the steel embedded in cement, such as pile caps and the steel in the soil. In the cell, the steel in the soil is the anode (corroding metal), and the steel in cement is the cathode (protected metal). This cell can be minimized by coating the part of the steel piles that will be embedded in cement to prevent contact with cement and reinforcing steel.

Piles driven into soils without disturbing soils will avoid oxygen introduction and low corrosion rates unless there is a probability for corrosive anaerobic bacteria. Galvanized steel's zinc coating can provide significant protection for driven piles. In corrosive soils in which normal zinc coatings are not enough, the life of piles can be extended by increasing zinc coating thickness, using sacrificial metal, or providing a combination of epoxy coatings and cathodic protection. Corrosion has been observed to be extremely localized even at and below underground water tables. Pit depths of this magnitude do not have an appreciable effect on the strength or useful life of piling structures because the reduction in pile cross section is not

[4] *Standard Requirements for Design and Analysis of Shallow Post-Tensioned Concrete Foundations on Expansive Soils, PTI DC10.5-12,Table 4.1, pg 16*
[5] *Specification for Unbonded Single Strand Tendons. Post-tensioning Institute (PTI), Phoenix, AZ, 2000.*

significant.[6] Pitting is of more importance to pipes transporting liquids or gases which should not be leaked into the ground.

The following recommendations are recommended to achieve desired life. We defer to structural engineers to use our estimated corrosion rates and to choose from the corrosion control options listed below.

1) Sacrificial metal by use of thicker piles per non-disturbed soil corrosion rates, or
2) Galvanized steel piles per non-disturbed soil corrosion rates, or
3) Combination of galvanized and sacrificial metal per non-disturbed soil corrosion rates, or
4) For no loss of metal, coat entire pile with abrasion resistant epoxy coating such as 3M Scotchkote 323, or PowercreteDD, or equivalent, or
5) Use high yield steel which will corrode at the same rate as mild steel but have greater yield strength and thus be able to suffer more material loss than mild steel.

2.5.1 Expected Corrosion Rate of Steel and Zinc in disturbed soil

In general, the corrosion rate of metals in soil depends on the electrical resistivity, the elemental composition, and the oxygen content of the soil. Soils can vary greatly from one acre to the next, especially at earthquake faults. The better a soil is for farming; the easier it will be for corrosion to take place. Expansive soils will also be considered disturbed simply because of their nature from dry to wet seasons.

In Melvin Romanoff's NBS Circular 579, the corrosion rates of carbon steels and various metals was studied over long term periods. Various metals were placed in various soil types to gather corrosion rate data of all metals in all soil types. Samples were collected and material loss measured over the course of 20 years in some sites. The following corrosion rates were estimated by comparing the worst results of soils tested with similar soils in Romanoff's studies and Highway Research Board's publications.[7] The corrosion rate of zinc in disturbed soils is determined per Romanoff studies and King Nomograph.[8]

Expected Corrosion Rate for Steel = 1.79 mils/year for one sided attack

Expected Corrosion Rate for Zinc = 0.08 mils/year for one sided attack.

Note: 1 mil = 0.001 inch

In undisturbed soils, a corrosion rate of 1 mil/year for steel is expected with little change in the corrosion rate of zinc due to it's low nobility in the galvanic series.

Per CTM 643: Years to perforation of corrugated galvanized steel culverts

- 29.1 Years to Perforation for a 18 gage metal culvert
- 37.8 Years to Perforation for a 16 gage metal culvert
- 46.5 Years to Perforation for a 14 gage metal culvert

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[6] Melvin Romanoff, Corrosion of Steel Pilings in Soils, National Bureau of Standards Monograph 58, pg 20.

[7] Field test for Estimating Service Life of Corrugated Metal Culverts, J.L. Beaton, Proc. Highway Research Board, Vol 41, P. 255, 1962

[8] King, R.A. 1977, Corrosion Nomograph, TRRC Supplementary Report, British Corrosion Journal

- 64.0 Years to Perforation for a 12 gage metal culvert
- 81.4 Years to Perforation for a 10 gage metal culvert
- 98.9 Years to Perforation for a 8 gage metal culvert

2.5.2 Expected Corrosion Rate of Steel and Zinc in Undisturbed soil

Expected Corrosion Rate for Steel = 1 mils/year for one sided attack

Expected Corrosion Rate for Zinc = 0.08 mils/year for one sided attack.

Note: 1 mil = 0.001 inch

2.6 Steel Storage tanks

Underground fuel tanks must be constructed and protected in accordance with California Underground Storage Tank Regulations, CCR, Title 23, Division 3, Chapter 16. Metals should be protected with cathodic protection or isolated from backfill material with an epoxy coating.

2.7 Steel Pipelines

Though a site may not be corrosive in nature at the time of construction, **installation of corrosion test stations and electrical continuity joint bonding should be performed during construction** so that future corrosion inspections can be performed. If steel pipes with gasket joints or other possibly non-conductive type joints are installed, their joints should be bonded across by welding or pin brazing a #8 AWG copper strand bond cable. Electrical continuity is necessary for corrosion inspections and for cathodic protection.

Corrosion test stations should be installed every 1,000 feet of pipeline.

Test stations shall have two #8 HMWPE copper strand wire test leads welded or pin brazed to the underground pipe, brought up into the test station hand hole and marked CTS. Wires should be brought into test station hand hole at finished grade with 12 inches of wire coiled within test station.

At isolation joints and pipe casings, 4 wire test stations shall be installed using #8 HMWPE copper strand wire test leads. Use different color wires to distinguish which wires are bonded to one side of isolation joint or to casing. Wires should be brought into test station hand hole at finished grade with 12 inches of wire coiled within test station.

Prevent dissimilar metal corrosion cells per NACE SP0286:

1) Electrically isolate dissimilar metal connections
2) Electrically isolate dissimilar coatings (Epoxy vs CML&C) segments connections
3) Electrically isolate river crossing segments
4) Electrically isolate freeway crossing segments
5) Electrically isolate old existing pipelines from new pipelines
6) Electrically isolate aboveground and underground pipe segments with flange isolation joint kits per NACE SP0286 to avoid galvanic corrosion cells. **These are especially important for fire risers.**

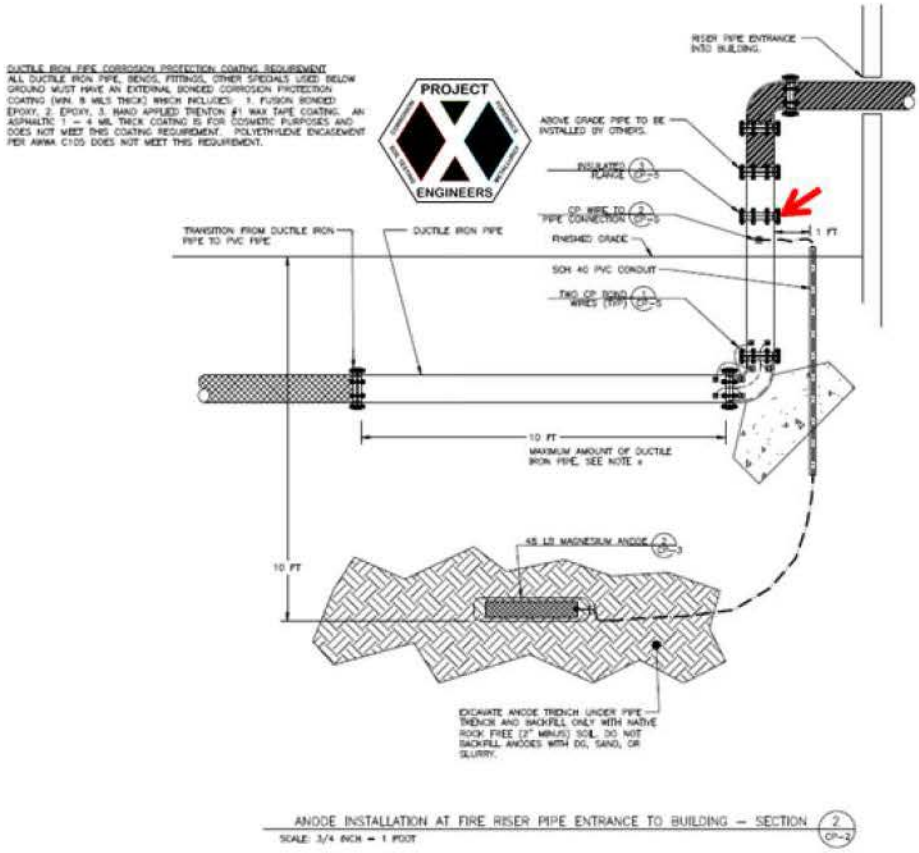

Figure 1- Fire Riser Detail: Install Isolation joint at red arrow

The bare steel surfaces, the corrosivity at this site is mildly corrosive to steel. The corrosion control options for this site are as follows:

1) Apply impermeable dielectric coating such as minimum 10 mil thick polyethylene, or
2) Tape coating system per AWWA C214, or
3) Wax tape per AWWA C217, or
4) Coal tar enamel per AWWA C203, or
5) Fusion bonded epoxy per AWWA C213, or
6) For bare steel surfaces, such as welded pipe joints, apply 3 inch thick field coating of Type II cement or high pH slurry that will maintain pH higher than 12. Cement is both a corrosion inhibitor and a coating for ferrous metals. Cement naturally holds a pH of 12 or higher for many years if not exposed to high levels of carbon dioxide. (For CML&C pipes, CML&C factory applied 3/4 inch thick coating is equivalent and needs no extra thickness added.)

It is critical for the life of the pipe that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

2.8 Steel Fittings

The corrosivity at this site is mildly corrosive to steel. The corrosion control options for this site can be one of the following:

1) Apply impermeable dielectric coating such as minimum 10 mil thick polyethylene, or
2) Tape coating system per AWWA C214, or
3) Wax tape per AWWA C217, or
4) Coal tar enamel per AWWA C203, or
5) Fusion bonded epoxy per AWWA C213
6) Use powder coated steel with minimum 60 micron (2-3 mil) thick coating[9], or
7) Galvanized steel, or
8) Apply standard concrete cover of Type II cement or high pH slurry that will maintain pH higher than 12. Cement is both a corrosion inhibitor and a coating for ferrous metals. Cement naturally holds a pH of 12 or higher for many years if not exposed to high levels of carbon dioxide.

It is critical for the life of the metal that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

2.9 Ductile Iron (DI) & Cast Iron Fittings

AWWA C105 developed a 10 point system to classify sites as aggressive or non-aggressive to ductile iron materials. The 10-point system does not, and was never intended to, quantify the corrosivity of a soil. It is a tool used to distinguish nonaggressive from aggressive soils relative to iron pipe. Soils <10 points are considered nonaggressive to iron pipe, whereas soils ≥ 10 points are considered aggressive. A 15 and a 20 point soil are both considered aggressive to iron pipe, however, because of the nature of the soil parameters measured, the 20 point soil may not necessarily be more aggressive than the 15 point soil. The criterion is based upon soil resistivities, soil drainage, pH, sulfide presence, and reduction-oxidation (REDOX) potential. The soil samples tested for this site resulted in a score of 6 out of 25.5. A score greater or equal

[9] Manish Kumar Bhadu, Akshya Kumar Guin, Veena Singh, Shyam K. Choudhary, "Corrosion Study of Powder-Coated Galvanised Steel", International Scholarly Research Notices, vol. 2013, Article ID 464710, 9 pages, 2013

to 10 points classifies soils as aggressive to iron materials. The black coating on iron pipes is purely for aesthetic purposes and should not be relied upon for corrosion protection.[10]

The corrosivity at this site is mildly corrosive to iron. The corrosion control options for this site are as follows:

1) Apply impermeable dielectric coating such as minimum 10 mil thick polyethylene, or
2) Tape coating system per AWWA C214, or
3) Wax tape per AWWA C217, or
4) Coal tar enamel per AWWA C203, or
5) Fusion bonded epoxy per AWWA C213
6) Apply standard concrete cover of Type II cement or high pH slurry that will maintain pH higher than 12. Cement is both a corrosion inhibitor and a coating for ferrous metals. Cement naturally holds a pH of 12 or higher for many years if not exposed to high levels of carbon dioxide.

It is critical for the life of the metal that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

2.10 Ductile Iron & Cast Iron Pipe

AWWA C105 developed a 10 point system to classify sites as aggressive or non-aggressive to ductile iron materials. The 10-point system does not, and was never intended to, quantify the corrosivity of a soil. It is a tool used to distinguish nonaggressive from aggressive soils relative to iron pipe. Soils <10 points are considered nonaggressive to iron pipe, whereas soils ≥10 points are considered aggressive. A 15 and a 20 point soil are both considered aggressive to iron pipe, however, because of the nature of the soil parameters measured, the 20 point soil may not necessarily be more aggressive than the 15 point soil. The criterion is based upon soil resistivities, soil drainage, pH, sulfide presence, and reduction-oxidation (REDOX) potential. The soil samples tested for this site resulted in a score of 6 out of 25.5. A score greater or equal to 10 points classifies soils as aggressive to iron materials. The black coating on iron pipes is purely for aesthetic purposes and should not be relied upon for corrosion protection.[11]

Though a site may not be corrosive in nature at the time of construction, **<u>installation of corrosion test stations and electrical continuity joint bonding should be performed during construction</u>** so that future corrosion inspections can be performed. If steel pipes with gasket joints or other possibly non-conductive type joints are installed, their joints should be bonded across by welding or pin brazing a #8 AWG copper strand bond cable. Electrical continuity is necessary for corrosion inspections and for cathodic protection. **If using thermite, perform one**

[10] https://www.dipra.org/ductile-iron-pipe-resources/frequently-asked-questions/corrosion-control

[11] https://www.dipra.org/ductile-iron-pipe-resources/frequently-asked-questions/corrosion-control

test bond using a half-charge then pressure test to confirm excess heat and pinholes were not created.

Pea gravel is used by plumbers to lay pipes and establish slopes. If the gravel has more than 200 ppm chlorides or is not tested, a 25 mil plastic should be placed between the gravel and pipe to avoid corrosion.

Corrosion test stations should be installed every 1,000 feet of pipeline.

Test stations shall have two #8 HMWPE copper strand wire test leads welded or pin brazed to the underground pipe, brought up into the test station hand hole and marked CTS. Wires should be brought into test station hand hole at finished grade with 12 inches of wire coiled within test station.

At isolation joints and pipe casings, 4 wire test stations shall be installed using #8 HMWPE copper strand wire test leads. Use different color wires to distinguish which wires are bonded to one side of isolation joint or to casing. Wires should be brought into test station hand hole at finished grade with 12 inches of wire coiled within test station.

Prevent dissimilar metal corrosion cells per NACE SP0286:

1) Electrically isolate dissimilar metal connections
2) Electrically isolate dissimilar coatings (Epoxy vs CML&C) segments connections
3) Electrically isolate river crossing segments
4) Electrically isolate freeway crossing segments
5) Electrically isolate old existing pipelines from new pipelines
6) Electrically isolate aboveground and underground pipe segments with flange isolation joint kits per NACE SP0286**. These are especially important for fire risers.**

The corrosivity at this site is mildly corrosive to iron. The corrosion control options for this site are as follows:

1) Apply impermeable dielectric coating such as minimum 10 mil thick polyethylene, or
2) Tape coating system per AWWA C214, or
3) Wax tape per AWWA C217, or
4) Coal tar enamel per AWWA C203, or
5) Fusion bonded epoxy per AWWA C213
6) Apply standard concrete cover of Type II cement or high pH slurry that will maintain pH higher than 12. Cement is both a corrosion inhibitor and a coating for ferrous metals. Cement naturally holds a pH of 12 or higher for many years if not exposed to high levels of carbon dioxide.

It is critical for the life of the metal that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

2.11 Copper Materials

Copper is an amphoteric material which is susceptible to corrosion at very high and very low pH. It is one of the most noble metals used in construction thus typically making it a cathode when connected to dissimilar metals. Copper's nobility can change with temperature, similar to the phenomenon in zinc. When zinc is at room temperature, it is less noble than steel and can provide cathodic protection to steel. But when zinc is at a temperature above 140F such as in a water heater, it becomes more noble than the steel and the steel becomes the sacrificial anode. This is why zinc is not used in steel water heaters or boilers. Cold copper has one native potential, but when heated it develops a more electronegative electro-potential aka open circuit potential. Thus hot and cold copper pipes should be electrically isolated from each other to avoid creation of a thermo-galvanic corrosion cell.

2.11.1 Copper Pipes

The lowest pH for this area was measured to be 6.3. Copper is greatly affected by pH, ammonia and nitrate concentrations[12]. The highest nitrate concentration was 41.6 mg/kg and the highest ammonia concentration was 43.2 mg/kg at this site.

These soils were determined to be corrosive to copper and copper alloys such as brass.

Aboveground, underground, cold water and hot water pipes should be electrically isolated from each other by use of dielectric unions and plastic in-wall pipe supports per NACE SP0286. The following are corrosion control options for underground copper water pipes.

1) Run copper pipes within PVC pipes to prevent soil contact, or
2) Cover piping with a 20 mil epoxy coating, or 8-mil polyethylene sleeve, or encase in double 4-mil thick polyethylene sleeves free of scratches and defects then backfill with clean sand with 2 inch minimum cover above and below tubing. Backfill should have a pH between 6 and 8 with electrical resistivity greater than 2,000 ohm-cm
3) Cover copper pipes with minimum 8 mil polyethylene sleeve or incase in double 4-mil thick polyethylene sleeves over a suitable primer and apply cathodic protection per NACE SP0169

It is critical for the life of the metal that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

[12] Corrosion Data Handbook, Table 6, Corrosion Resistance of copper alloys to various environments, 1995

2.11.2 Brass Fittings

Brass fittings should be electrically isolated from dissimilar metals by use of dielectric unions or isolation joint kits per NACE SP0286.

These soils were determined to be corrosive to copper and copper alloys such as brass.

The following are corrosion control options for underground brass.

1) Prevent soil contact by use of impermeable coating system such as wax tape, or
2) Prevent soil contact by use of a 20 mil epoxy coating free of scratches and defects and backfill with clean sand with 4 inch minimum cover above and below brass. Backfill should have a pH between 6 and 8 with electrical resistivity greater than 2,000 ohm-cm, or
3) Cover brass with minimum 10 mil polyethylene sleeve over a suitable primer and apply cathodic protection per NACE SP0169

It is critical for the life of the metal that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

2.11.3 Bare Copper Grounding Wire

It is assumed that corrosion will occur at all sides of the bare wire, thus the corrosion rate is calculated as a two sided attack determining the time it takes for the corrosion from two sides to meet at the center of the wire. The estimated life of bare copper wire for this site is the following:[13]

Size (AWG)	Diameter (mils)	Est. Time to penetration (Yrs)
14	64.1	5.5
13	72	6.2
12	80.8	7.0
11	90.7	7.8
10	101.9	8.8
9	114.4	9.9
8	128.5	11.1
7	144.3	12.4
6	162	14.0
5	181.9	15.7
4	204.3	17.6
3	229.4	19.8
2	257.6	22.2

[13] Soil-Corrosion studies 1946 and 1948: Copper Alloys, Lead, and Zinc, Melvin Romanoff, National Bureau of Standards, Research Paper RP2077, 1950

Size (AWG)	Diameter (mils)	Est. Time to penetration (Yrs)
1	289.3	24.9

If the bare copper wire is being used as a grounding wire connected to less noble metals such as galvanized steel or carbon steel, the less noble metals will provide additional cathodic protection to the copper reducing the corrosion rate of the copper.

It is recommended that a corrosion inhibiting and water-repelling coating be applied to aboveground and belowground copper-to-dissimilar metal connections to reduce risk of dissimilar corrosion. This can be wax tape, or other epoxy coating.

Tinned copper wiring or laying copper wire in conductive concrete can protect against chemical attack in soils with high nitrates, ammonia, sulfide and severely low soil electrical resistivity.

2.12 Aluminum Pipe/Conduit/Fittings

Aluminum is an amphoteric material prone to pitting corrosion in environments that are very acidic or very alkaline or high in chlorides.

Conditions at this site are safe for aluminum.

Aluminum derives its corrosion resistance from its oxide layer which needs oxygen to regenerate if damaged, similar to stainless steels. Thus aluminum is not good for deep soil applications. Since aluminum corrodes at very alkaline environments, it cannot be encased or placed against cement or mortar such as brick wall mortar up against an aluminum window frame.

Aluminum is also very low on the galvanic series scale making it most likely to become a sacrificial anode when in contact with dissimilar metals in moist environments. Avoid electrical continuity with dissimilar metals by use of insulators, dielectric unions, or isolation joints per NACE SP0286. Pooling of water at post bottoms or surfaces should be avoided by integrating good drainage.

2.13 Carbon Fiber or Graphite Materials

Carbon fiber or other graphite materials are extremely noble on the galvanic series and should always be electrically isolated from dissimilar metals. They can conduct electricity and will create corrosion cells if placed in contact within a moist environment with any metal.

2.14 Plastic and Vitrified Clay Pipe

No special precautions are required for plastic and vitrified clay piping from a corrosion viewpoint.

Protect all metallic fittings and pipe restraining joints with wax tape per AWWA C217, cement if previously recommended, or epoxy.

3 CLOSURE

In addition to soils chemistry and resistivity, another contributing influence to the corrosion of buried metallic structures is stray electrical currents. These electrical currents flowing through the earth originate from buried electrical systems, grounding of electrical systems in residences, commercial buildings, and from high voltage overhead power grids. Therefore, it is imperative that the application of protective wraps and/or coatings and electrical isolation joints be properly applied and inspected.

It is the responsibility of the builder and/or contractor to closely monitor the installation of such materials requiring protection in order to assure that the protective wraps or coatings are not damaged.

The recommendations outlined herein are in conformance with current accepted standards of practice that meet or exceed the provisions of the Uniform Building Code (UBC), the International Building Code (IBC), California Building Code (CBC), the American Cement Institute (ACI), Nickel Institute, National Association of Corrosion Engineers (NACE International), Post-Tensioning Institute Guide Specifications and State of California Department of Transportation, Standard Specifications, American Water Works Association (AWWA) and the Ductile Iron Pipe Research Association (DIPRA).

Our services have been performed with the usual thoroughness and competence of the engineering profession. No other warranty or representation, either expressed or implied, is included or intended.

Please call if you have any questions.

Respectfully Submitted,

Ed

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EDUARDO HERNANDEZ
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MECHANICAL
STATE OF CALIFORNIA

4 SOIL ANALYSIS LAB RESULTS

Client: Geotechnologies, Inc.
Job Name: Chandler Partners
Client Job Number: 22119
Project X Job Number: S220209E
February 11, 2022

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM G51	ASTM G200	SM 4500-D	ASTM D4327	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D4327	ASTM D4327
Bore# / Description	**Depth**	**Sulfates** SO_4^{2-}		**Chlorides** Cl^-		**Resistivity** As Rec'd	Minimum	**pH**	**Redox**	**Sulfide** S^{2-}	**Nitrate** NO_3^-	**Ammonium** NH_4^+	**Lithium** Li^+	**Sodium** Na^+	**Potassium** K^+	**Magnesium** Mg^{2+}	**Calcium** Ca^{2+}	**Fluoride** F_2^-	**Phosphate** PO_4^{3-}
	(ft)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(Ohm-cm)	(Ohm-cm)		(mV)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)
B1 SM	2.5	40.7	0.0041	3.8	0.0004	9,380	3,149	7.3	131	1.44	39.1	10.9	0.04	45.2	16.0	33.9	41.4	0.6	3.1
B2 SM	2.5	127.4	0.0127	13.3	0.0013	10,720	4,757	7.6	142	2.22	19.5	43.2	ND	18.6	6.5	22.1	57.7	4.4	12.2
B4 SM	2.5	78.3	0.0078	27.3	0.0027	542,700	3,417	7.2	146	0.84	41.6	13.7	0.07	86.1	31.0	16.8	34.7	7.4	1.2
B5 SM	2.5	325.6	0.0326	4.7	0.0005	>737,000	1,072	7.3	130	0.33	2.9	17.3	0.02	12.2	6.1	13.6	31.2	4.8	39.3
TP1 SM	3	56.9	0.0057	23.7	0.0024	>737,000	7,370	6.3	160	1.05	5.9	24.8	0.01	160.0	12.3	13.0	15.9	6.4	3.9
TP3 SM/SP	3	122.2	0.0122	36.3	0.0036	>737,000	8,040	7.2	169	0.15	4.6	6.4	0.04	41.0	3.6	16.3	23.9	3.7	1.0

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

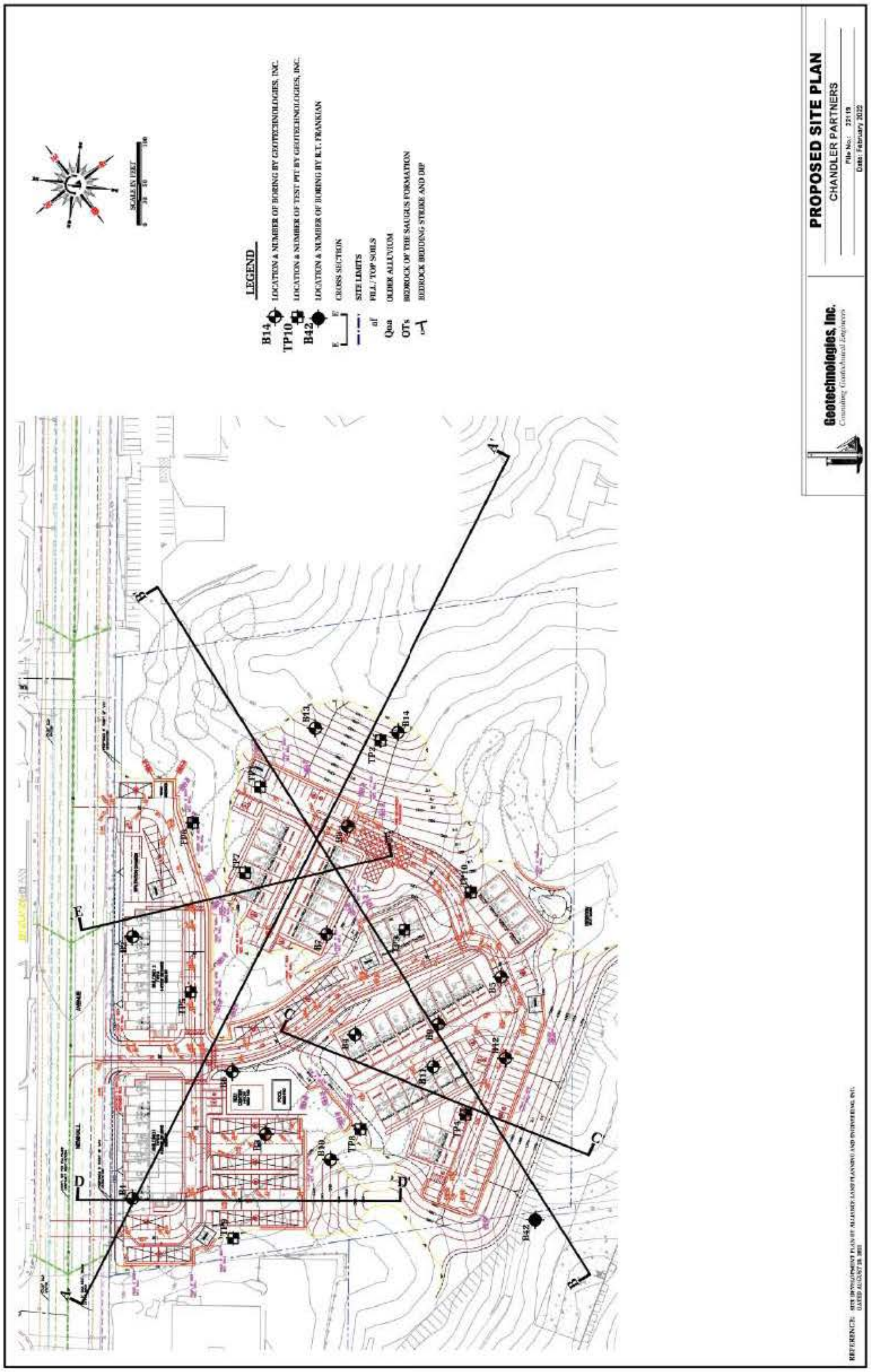


Figure 2- Soil Sample Locations, 23755 Newhall Avenue, Santa Clarita, California

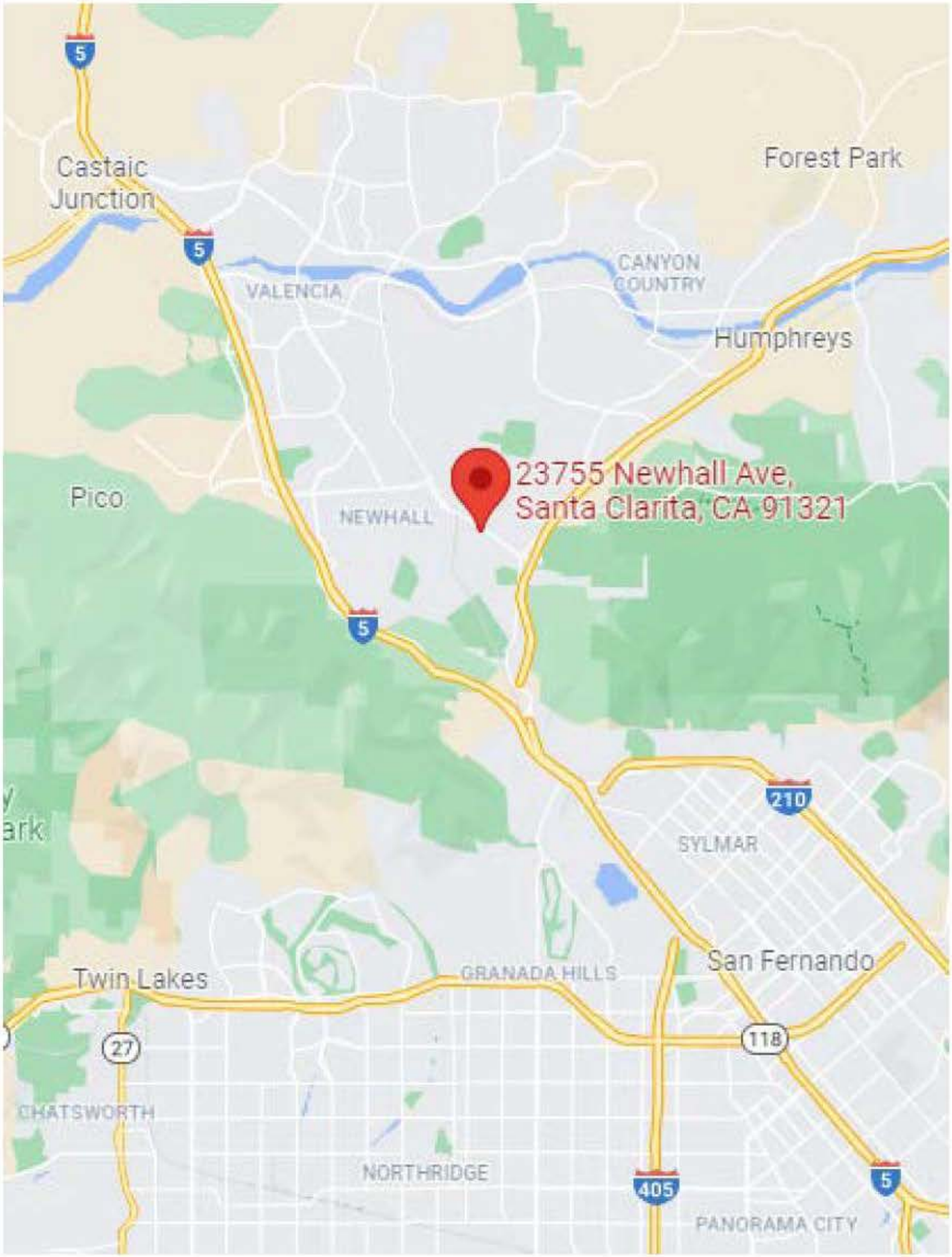


Figure 3- Vicinity Map, 23755 Newhall Avenue, Santa Clarita, California

5 Corrosion Basics

In general, the corrosion rate of metals in soil depends on the electrical resistivity, the elemental composition, and the oxygen content of the soil. Soils can vary greatly from one acre to the next, especially at earthquake faults. The better a soil is for farming; the easier it will be for corrosion to take place. Expansive soils should be considered disturbed simply because of their nature from dry to wet seasons.

5.1 Pourbaix Diagram – In regards to a material's environment

All metals are unique and have a weakness. Some metals do not like acidic (low pH) environments. Some metals do not like alkaline (high pH) environments. Some metals don't like either high or low pH environments such as aluminum. These are called amphoteric materials. Some metals become passivated and do not corrode at high pH environments such as steel. These characteristics are documented in Marcel Pourbaix's book "Atlas of electrochemical equilibria in aqueous solutions"

In the mid 1900's, Marcel Pourbaix developed the Pourbaix diagram which describes a metal's reaction to an environment dependent on pH and voltage conditions. It describes when a metal remains passive (non-corroding) and in which conditions metals become soluble (corrode). Steels are passive in pH over 12 such as the condition when it is encased in cement. If the cement were to carbonate and its pH reduce to below 12, the cement would no longer be able to act as a corrosion inhibitor and the steel will begin to corrode when moist.

Some metals such as aluminum are amphoteric, meaning that they react with acids and bases. They can corrode in low pH and in high pH conditions. Aluminum alloys are generally passive within a pH of 4 and 8.5 but will corrode outside of those ranges. This is why aluminum cannot be embedded in cement and why brick mortar should not be laid against an aluminum window frame without a protective barrier between them.

5.2 Galvanic Series – In regards to dissimilar metal connections

All metals have a natural electrical potential. This electrical potential is measured using a high impedance voltmeter connected to the metal being tested and with the common lead connected to a copper copper-sulfate reference electrode (CSE) in water or soil. There are many types of reference electrodes. In laboratory measurements, a Standard Hydrogen Electrode (SHE) is commonly used. When different metal alloys are tested they can be ranked into an order from most noble (less corrosion), to least noble (more active corrosion). When a more noble metal is connected to a less noble metal, the less noble metal will become an anode and sacrifice itself through corrosion providing corrosion protection to the more noble metal. This hierarchy is known as the galvanic series named after Luigi Galvani whose experiments with electricity and muscles led Alessandro Volta to discover the reactions between dissimilar metals leading to the early battery. The greater the voltage difference between two metals, the faster the corrosion rate will be.

Table 1- Dissimilar Metal Corrosion Risk

	Zinc	Galvanized Steel	Aluminum	Cast Iron	Lead	Mild Steel	Tin	Copper	Stainless Steel
Zinc	None	Low	Medium	High	High	High	High	High	High
Galvanized Steel	Low	None	Medium	Medium	Medium	High	High	High	High
Aluminum	Medium	Medium	None	Medium	Medium	Medium	Medium	High	High
Cast Iron	High	Medium	Medium	None	Low	Low	Low	Medium	Medium
Lead	High	Medium	Medium	Low	None	Low	Low	Medium	Medium
Mild Steel	High	High	Medium	Low	Low	None	Low	Medium	Medium
Tin	High	High	Medium	Low	Low	Low	None	Medium	Medium
Copper	High	High	High	Medium	Medium	Medium	Medium	None	Low
Stainless Steel	High	High	High	Medium	Medium	Medium	Medium	Low	None

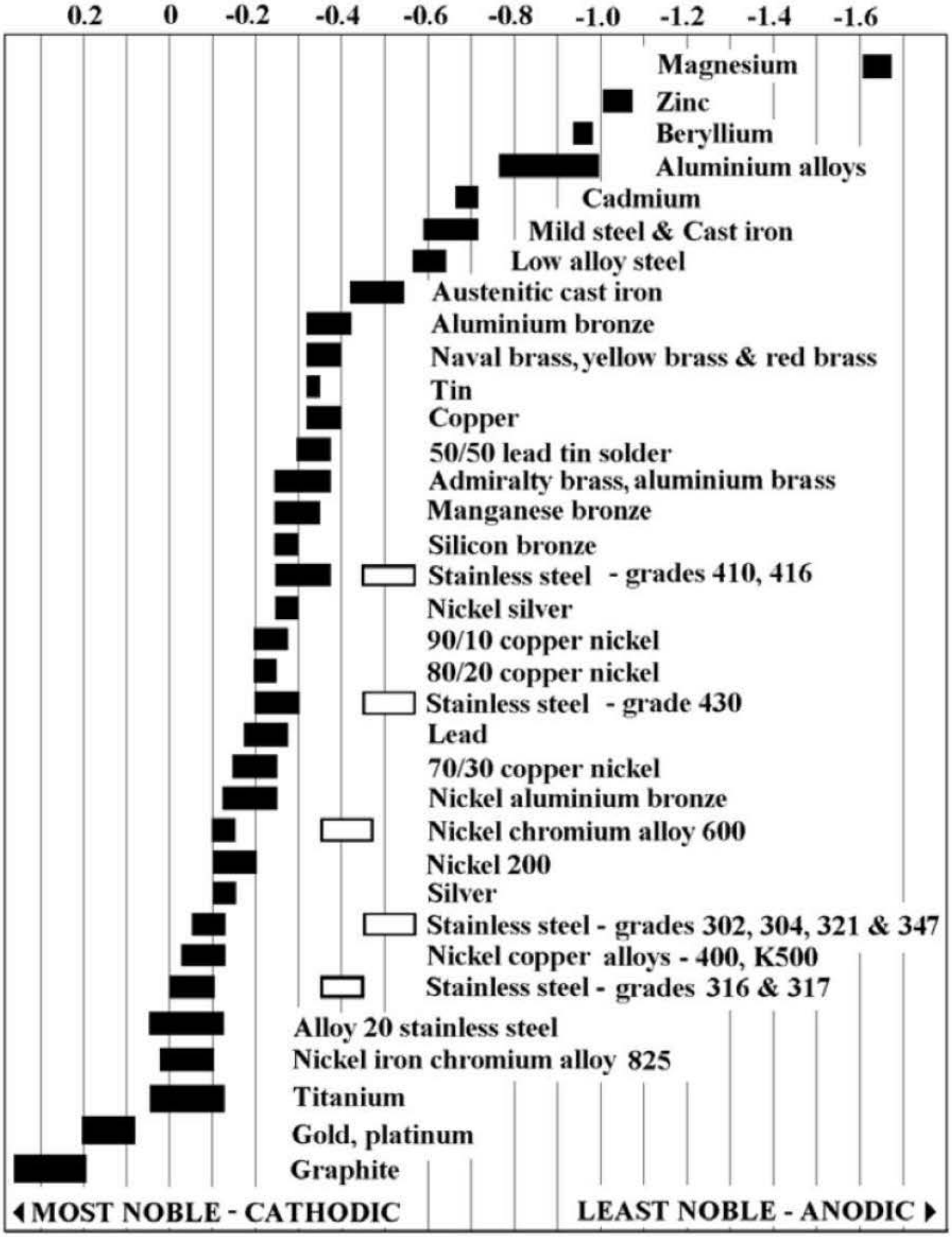
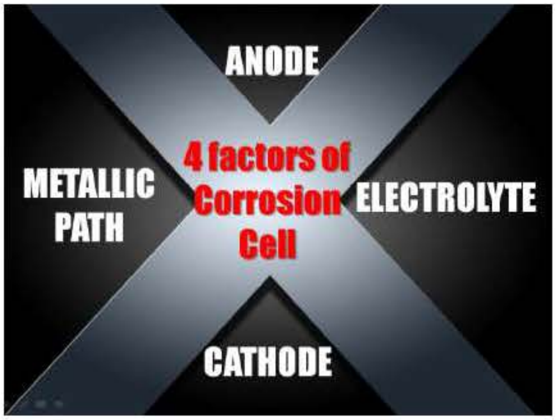


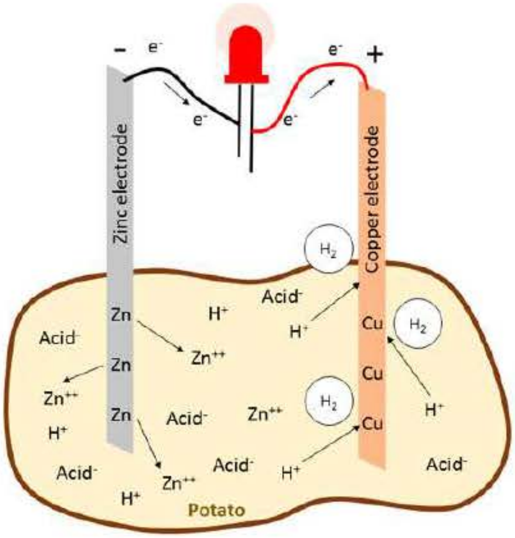
Figure 4 - Galvanic series of metals relative to CSE half cell.

5.3 Corrosion Cell

In order for corrosion to occur, four factors must be present. (1) The anode (2) the cathode (3) the electrolyte and (4) the metallic or conductive path joining the anode and the cathode. If any one of these is removed, corrosion activity will stop. This is how a simple battery produces electricity. An example of a non-metallic yet conductive material is graphite. Graphite is similar in nobility to gold. Do not connect graphite to anything in moist environments.



The anode is where the corrosion occurs, and the cathode is the corrosion free material. Sometimes the anode and cathode are different materials connected by a wire or union. Sometimes the anode and cathode are on the same pipe with one area of the pipe in a low oxygen zone while the other part of the pipe is in a high oxygen zone. A good example of this is a post in the ocean that is repeatedly splashed. Deep underwater, corrosion is minimal, but at the splash zone, the corrosion rate is greatest.



Low oxygen zones and crevices can also harbor corrosive bacteria which in moist environments will lead to corrosion. This is why pipes are laid on backfill instead of directly on native cut soil in a trench. Filling a trench slightly with backfill before installing pipe then finishing the backfill creates a uniform environment around the entire surface of the pipe.

The electrolyte is generally water, seawater, or moist soil which allows for the transfer of ions and electrical current. Pure water itself is not very conductive. It is when salts and minerals dissolve into pure water that it becomes a good conductor of electricity and chemical reactions. Metal ores are turned into metal alloys which we use in construction. They naturally want to return to their natural metal ore state but it requires energy to return to it. The corrosion cell, creates the energy needed to return a metal to its natural ore state.

The metallic or conductive path can be a wire or coupling. Examples are steel threaded into a copper joint, or an electrician grounding equipment to steel pipes inadvertently connecting electrical grid copper grounding systems to steel or iron underground pipes.

The ratio of surface area between the anode and the cathode is very important. If the anode is very large, and the cathode is very small, then the corrosion rate will be very small and the anode may live a long life. An example of this is when short copper laterals were connected to a large and long steel pipeline. The steel had plenty of surface area to spread the copper's attack, thus corrosion was not noticeable. But if the copper was the large pipe and the steel the short laterals, the steel would corrode at an amazing rate.

5.4 Design Considerations to Avoid Corrosion

The following recommendations are based upon typical observations and conclusions made by forensic engineers in construction defect lawsuits and NACE International (Corrosion Society) recommendations.

5.4.1 Testing Soil Factors (Resistivity, pH, REDOX, SO, CL, NO3, NH3)

As previously mentioned, different factors can cause corrosion. The most useful and common test for categorizing a soil's corrosivity has been the measure of soil resistivity which is typically measured in units of (ohm-cm) by corrosion engineers and geologists. Soil resistivity is the ability of soil to conduct or resist electrical currents and ion transfer. The lower the soil resistivity, the more conductive and corrosive it is. The following are "generally" accepted categories but keep in mind, the question is not "Is my soil corrosive?", the question should be, "What is my soil corrosive to?" and to answer that question, soil resistivity and chemistry must be tested. Though **soil resistivity is a good corrosivity indicator for steel materials, high chlorides or other corrosive elements do not always lower soil resistivity, thus if you don't test for chlorides and other water soluble salts, you can get an unpleasant surprise.** The largest contributing factor to a soil's electrical resistivity is its clay, mineral, metal, or sand make-up.

Table 2 - Corrosion Basics- An Introduction, NACE, 1984, pg 191

(Ohm-cm)	Corrosivity Description
0-500	Very Corrosive
500-1,000	Corrosive
1,000-2,000	Moderately Corrosive
2,000-10,000	Mildly Corrosive
Above 10,000	Progressively less corrosive

Testing a soil's pH provides information to reference the Pourbaix diagram of specific metals. Some elements such as ammonia and nitrates can create localized alkaline conditions which will greatly affect amphoteric materials such as aluminum and copper alloys.

Excess sulfates can break-down the structural integrity of cement and high concentrations of chlorides can overcome cement's corrosion inhibiting effect on encased ferrous metals and break down protective passivated surface layers on stainless steels and aluminum.

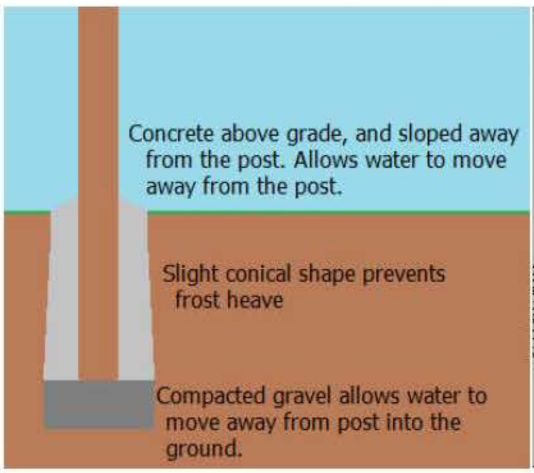
Corrosive bacteria are everywhere but can multiply significantly in anaerobic conditions with plentiful sulfates. The bacteria themselves do not eat the metal but their by-products can form corrosive sulfuric acids. The probability of corrosive bacteria is tested by measuring a soil's oxidation-reduction (REDOX) electro-potential and by testing for the presence of sulfides.

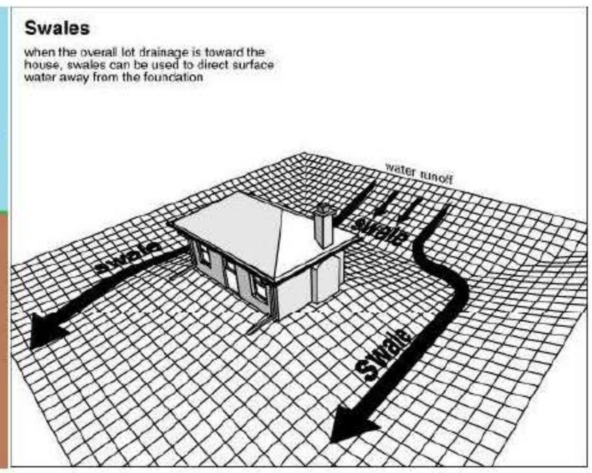
Only by testing a soil's chemistry for minimum resistivity, pH, chlorides, sulfates, sulfides, ammonia, nitrate, and redox potential can one have the information to evaluate the corrosion risk to construction materials such as steel, stainless steel, galvanized steel, iron, copper, brass, aluminum, and concrete.

5.4.2 Proper Drainage

It cannot be emphasized enough that pooled stagnant water on metals will eventually lead to corrosion. This stands for internal corrosion and external corrosion situations. In soils, providing good drainage will lower soil moisture content reducing corrosion rates. Attention to properly sealing polyethylene wraps around valves and piping will avoid water intrusion which would allow water to pool against metals. Above ground structures should not have cupped or flat surfaces that will pond water after rain or irrigation events.

Buildings typically are built on pads and have swales when constructed to drain water away from buildings directing it towards an acceptable exit point such as a driveway where it continues draining to a local storm drain. Many homeowners, landscapers and flatwork contractors appear to not be aware of this and destroy swales during remodeling. The majority of garage floor and finished grade elevations are governed by drainage during design.[14,15]





5.4.3 Avoiding Crevices

Crevices are excellent locations for oxygen differential induced corrosion cells to begin. Crevices can also harbor corrosive bacteria even in the most chemically treated waters. Crevices will also gather salts. If water's total alkalinity is low, its ability to maintain a stable pH can also become more difficult within a crevice allowing the pH to drop to acidic levels continuing a pitting process. Welds in extremely corrosive environments should be complete and well filleted without sharp edges to avoid crevices. Sharp edges should be avoided to allow uniform coating of protective epoxy. Detection of crevices in welds should be treated immediately. If pressures and loads are low, sanding and rewelding or epoxy patching can be suitable repairs. Damaged coatings can usually be repaired with Direct to Metal paints. **Scratches and crevice corrosion are like infections, they should not be left to fester or the infection will spread making things worse.**

[14] https://www.fencedaddy.com/blogs/tips-and-tricks/132606467-how-to-repair-a-broken-fence-post

[15] http://southdownstudio.co.uk/problme-drainage-maison.html

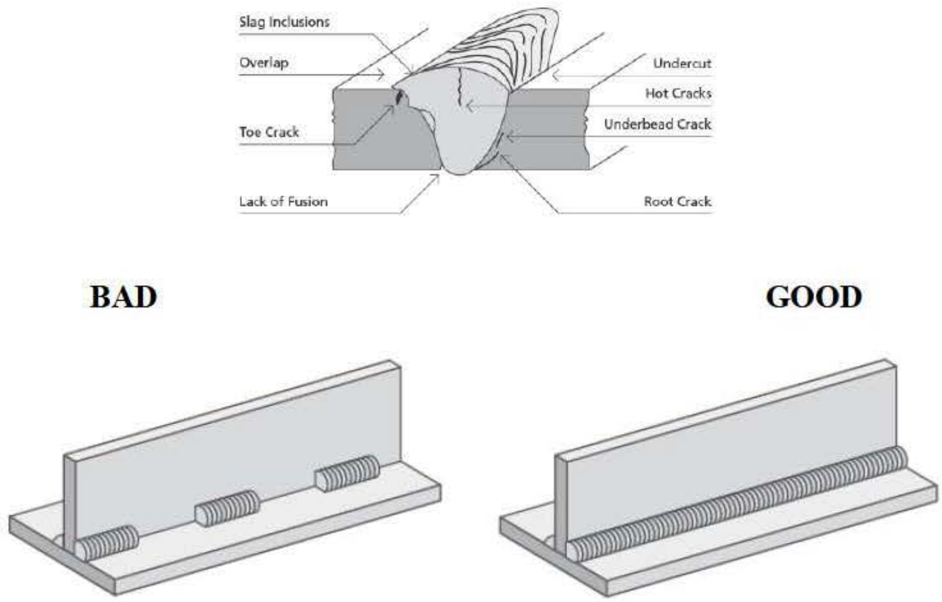


Figure 5 Defects which form weld crevices[16]

5.4.4 Coatings and Cathodic Protection

When faced with a corrosive environment, the best defense against corrosion is removing the electrolyte from the corrosion cell by applying coatings to separate the metal from the soil. During construction and installation, there is always some scratch or damage made to a coating. NACE training recommends that coatings be used as a first line of defense and that sacrificial or impressed current cathodic protection is used as a 2nd line of defense to protect the scratched areas. Use of a good coating dramatically reduces the amount of anodes a CP system would need. If CP is not installed as a 2nd line of defense in an extremely corrosive environment, the small scratched zones will suffer accelerated corrosion. CP details such as anode installation instructions must be designed by corrosion engineers or vessel manufacturers on a per project basis because it depends on electrolyte resistivity, surface area of infrastructure to be protected, and system geometry.

There are two types of cathodic protection systems, a Galvanic Anode Cathodic Protection (GACP) system and an Impressed Current Cathodic Protection (ICCP) system. A Galvanic Anode Cathodic Protection (GACP) system is simpler to install and maintain than an Impressed Current Cathodic Protection (ICCP) system. To protect the metals, they must all be electrically continuous to each other. In a GACP system, sacrificial zinc or magnesium anodes are then buried at locations per the CP design and connected by wire to a structure at various points in system. At the connection points, a wire connecting to the structure and the wire from the anode are joined in a Cathodic Protection Test Station hand hole which looks similar in size and shape to an irrigation valve pull box. By coating the underground structures, one can reduce the number of anodes needed to provide cathodic protection by 80% in many instances.

An ICCP system requires a power source, a rectifier, significantly more trenching, and more expensive type anodes. These systems are typically specified when bare metal is requiring protection

[16] http://www.daroproducts.co.uk/makes-good-weld/

in severely corrosive environments in which galvanic anodes do not provide enough power to polarize infrastructure to -850 mV structure-to-soil potential or be able to create a 100 mV potential shift as required by NACE SP169 to control corrosion. In severely corrosive environments, a GACP system simply may not last a required lifetime due to the high rate of consumption of the sacrificial anodes. ICCP system rectifiers must be inspected and adjusted quarterly or at a minimum bi-annually per NACE recommendations. Different anode installations may be possible but for large sites, anodes are placed evenly throughout the site and all anode wires must be trenched to the rectifier. For a large site, it may be beneficial to use two or more rectifiers to reduce wire lengths or trenching.

To simplify, a GACP system can be installed and practically forgotten with minor trenching because the anodes can be installed very close to the structures. An ICCP system must be inspected annually and anode wires run back to the rectifier which itself connects to the pile system. If any type of trenching or development is expected to occur at the site during the life of the site, it is a good idea to inspect the anode connections once a year to make sure wires are not cut and that the infrastructure is still being provided adequate protection. A common situation that occurs with ICCP systems is that a contractor accidently cuts the wires during construction then reconnects them incorrectly, turning the once cathode, into a sacrificing anode.

Design of a cathodic protection system protecting against soil side corrosion requires that Wenner Four Pin ground resistance measurements per ASTM G57 be performed by corrosion engineers at various locations of the site to determine the best depths and locations for anode installations. Ideally, a sample pile is installed and experiments determining current requirement are conducted. Using this data, the decision is made whether a GACP system is feasible or if an ICCP must be used.

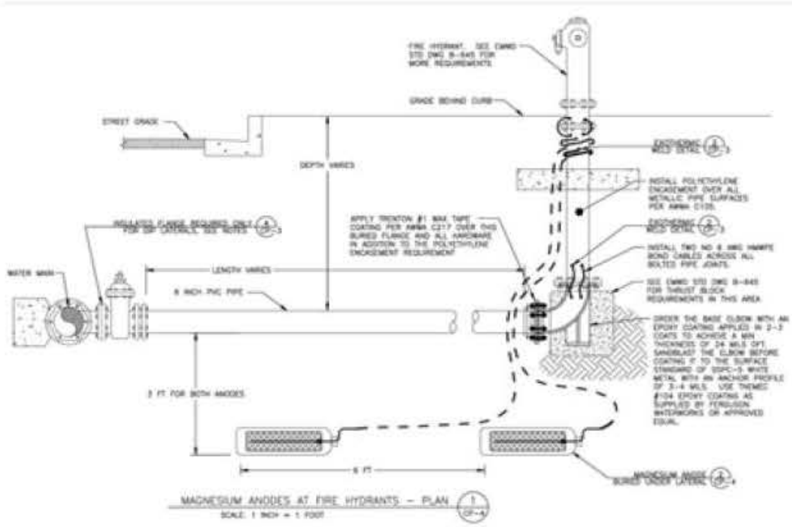


Figure 6 Sample anode design for fire hydrant underground piping

Vessels such as water tanks will have protective interior coatings and anodes to protect the interior surfaces. Anodes can also be buried on site and connected to system skid supports to protect the metal in contact with soil. A good example of a vessel cathodic protection system exists in all home water heaters which contain sacrificial aluminum or magnesium anodes. In environments that exceed 140F, zinc anodes cannot be used with carbon steel because they become the aggressor (Cathodic) to the steel instead of sacrificial (anodic). Anodes in vessels containing extremely brackish water with chloride levels over 2,000 ppm should inspect or change out their anodes every 6 months.

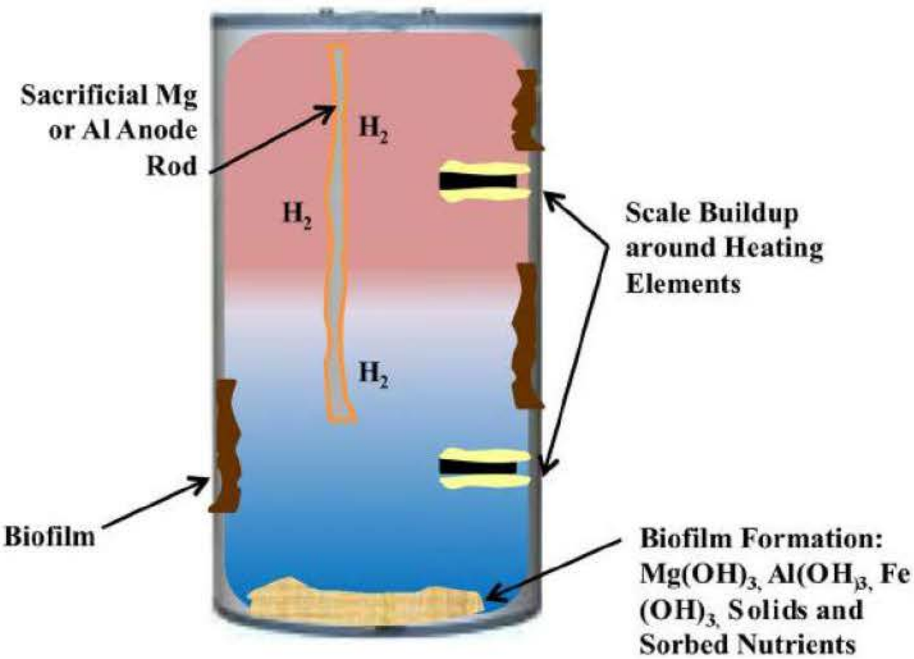


Figure 7 Cross section of boiler with anode

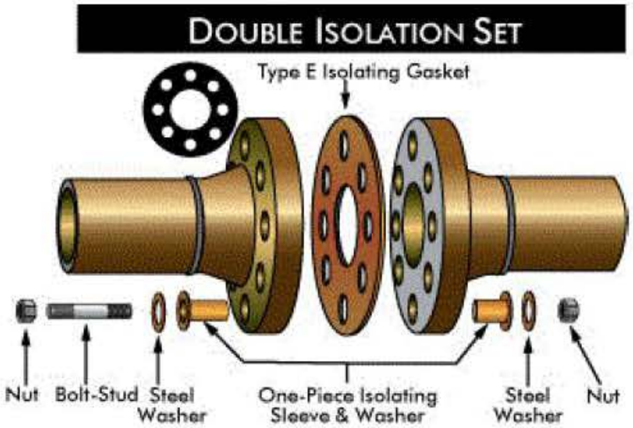
Cathodic protection can only protect a few diameters within a pipeline thus it is not recommended for small diameter pipelines and tubing internal corrosion protection. Anodes are like a lamp shining light in a room. They can only protect along their line of sight.

5.4.5 Good Electrical Continuity

In order for cathodic protection to protect a long pipeline or system of pipes from external soil side corrosion, they must all be electrically continuous to each other so that the electric current from the anode can travel along the pipes, then return through the earth to the anode. Electrical continuity is achieved by welding or pin brazing #8 AWG copper strand bond cable to the end of pipe sticks which have rubber gaskets at bell and spigots. If steel pipes are joined by full weld, bonding wires are not needed.

Electrical continuity between dissimilar metals is not desirable. Isolation joints or di-electric unions should be installed between dissimilar metals, such as steel pipes connecting to a brass valve per NACE SP0286. Bonding wires should then be welded onto the steel pipes by-passing the brass valve so that the cathodic protection system's current can continue to travel along the steel piping but isolate the brass valve from the steel pipeline. Another option would be to provide a separate cathodic protection system for steel pipes on both sides of the brass valve.

Typically, water heater inlets and outlets, gas meters and water meters have dielectric unions installed in them to separate utility property from homeowner property. This also protects them in the case that a home owner somehow electrically connects water pipes or gas pipes to a neighborhood electrical grounding system which can potentially have less noble steel in soil now connected to much more noble copper in soil which will then create a corrosion cell. This is exactly how a lemon powered clock works when a galvanized zinc nail and a steel nail are inserted into a lemon then connected to a clock. The clock is powered by the corrosion cell created.



5.4.6 Bad Electrical Continuity

Bad electrical continuity is when two different materials or systems are made electrically continuous (aka shorted) when they were not designed to be electrically continuous. Examples of this would be when gas lines are shorted to water lines or to electrical grounding beds. Very often, fire risers are shorted to electrical grounding systems, and water pipes at business parks. Since fire risers usually have a very short ductile iron pipe in the ground which connects to PVC pipe systems, they tend to experience leaks after 7 to 10 years of being attacked by underground copper systems.

It is absolutely imperative that any copper water piping or other metal conduits penetrating cement slab or footings, not come in contact with the reinforcing steel or post-tensioning tendons to avoid creation of galvanic corrosion cells.

5.4.7 Corrosion Test Stations

Corrosion test stations should be installed every 1,000 feet along pipelines in order to measure corrosion activity in the future. For a simple pipeline, two #8 AWG copper strand bond cable welded or pin brazed onto the pipeline are run up to finished grade and left in a hand hole. Corrosion test stations are used to measure pipe-to-soil electro potential relative to a copper copper-sulfate reference electrode to determine if the pipe is experiencing significant corrosion activity. By measuring test stations along a pipeline, hot spots can be determined, if any. The wires also allow for electrical continuity testing, condition assessment, and a multitude of other types of tests.

At isolation joints and pipe casings, two wires should be welded to either side of the isolation joint for a total of 4 wires to be brought up to the hand hole. This allows for future tests of the isolation joint, casing separation confirmation, and pipe-to-soil potential readings during corrosion surveys.

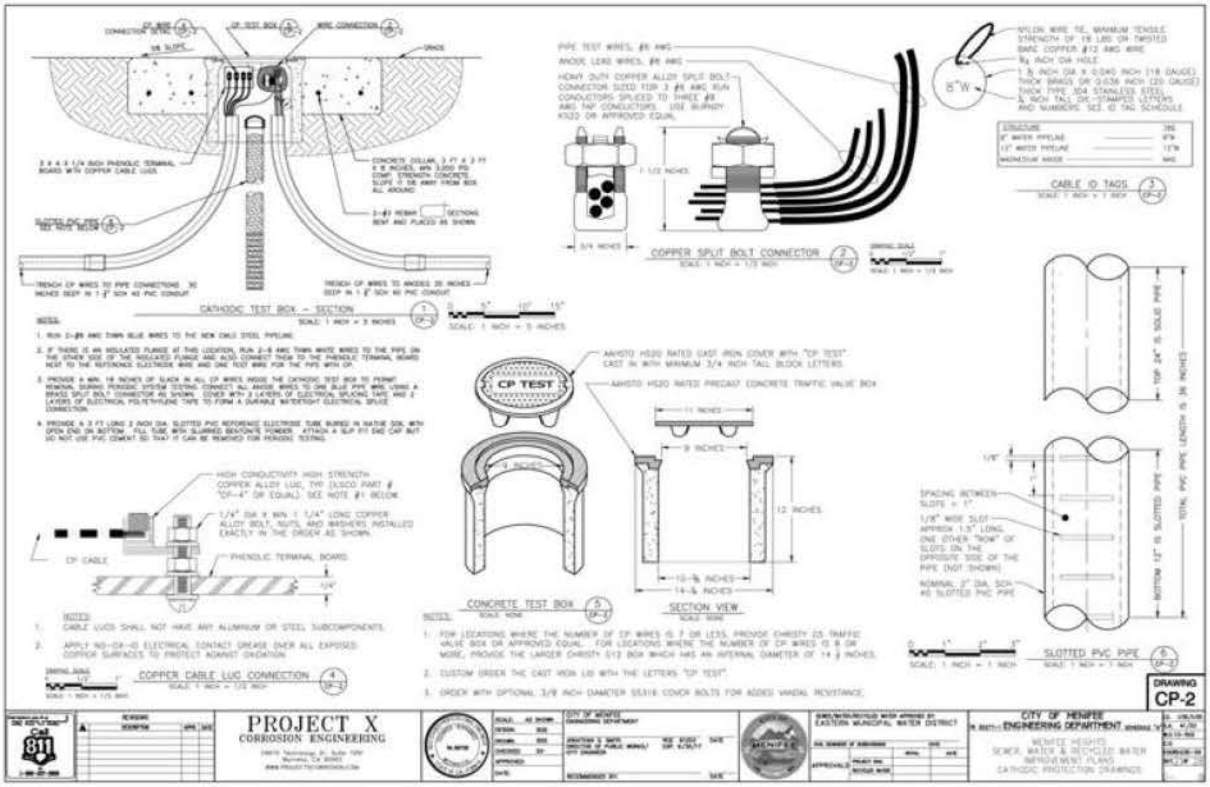


Figure 8 Sample of corrosion test station specification drawing

5.4.8 Excess Flux in Plumbing

Investigations of internal corrosion of domestic water plumbing systems almost always finds excess flux to be the cause of internal pitting of copper pipes. Some people believe that there is no such thing as too much flux. Flux runs have been observed to travel up to 20 feet with pitting occurring along the flux run. Flushing a soldered plumbing system with hot water for 15 minutes can remove significant amounts of excess flux left in the pipes. If a plumbing system is expected to be stagnant for some time, it should be drained to avoid stagnant water conditions that can lead to pitting and dezincification of yellow brasses.

5.4.9 Landscapers and Irrigation Sprinkler Systems

A significant amount of corrosion of fences is due to landscaper tools scratching fence coatings and irrigation sprinklers spraying these damaged fences. Recycled water typically has a higher salt content than potable drinking water, meaning that it is more corrosive than regular tap water. The same risk from damage and water spray exists for above ground pipe valves and backflow preventers. Fiber glass covers, cages, and cement footings have worked well to keep tools at an arm's length.

5.4.10 Roof Drainage splash zones

Unbelievably, even the location where your roof drain splashes down can matter. We have seen drainage from a home's roof valley fall directly down onto a gas meter causing it's piping to corrode at an accelerated rate reaching 50% wall thickness within 4 years. It is the same effect as a splash zone in the ocean or in a pool which has a lot of oxygen and agitation that can remove material as it corrodes.

5.4.11 Stray Current Sources

Stray currents which cause material loss when jumping off of metals may originate from direct-current distribution lines, substations, or street railway systems, etc., and flow into a pipe system or other steel structure. Alternating currents may occasionally cause corrosion. The corrosion resulting from stray currents (external sources) is similar to that from galvanic cells (which generate their own current) but different remedial measures may be indicated. In the electrolyte and at the metal-electrolyte interfaces, chemical and electrical reactions occur and are the same as those in the galvanic cell; specifically, the corroding metal is again considered to be the anode from which current leaves to flow to the cathode. Soil and water characteristics affect the corrosion rate in the same manner as with galvanic-type corrosion.

However, stray current strengths may be much higher than those produced by galvanic cells and, as a consequence, corrosion may be much more rapid. Another difference between galvanic-type currents and stray currents is that the latter are more likely to operate over long distances since the anode and cathode are more likely to be remotely separated from one another. Seeking the path of least resistance, the stray current from a foreign installation may travel along a pipeline causing severe corrosion where it leaves the line. Knowing when stray currents are present becomes highly important when remedial measures are undertaken since a simple sacrificial anode system is likely to be ineffectual in preventing corrosion under such circumstances.[17] Stray currents can be avoided by installing proper electrical shielding, installation of isolation joints, or installation of sacrificial jump off anodes at crossings near protected structures such as metal gas pipelines or electrical feeders.

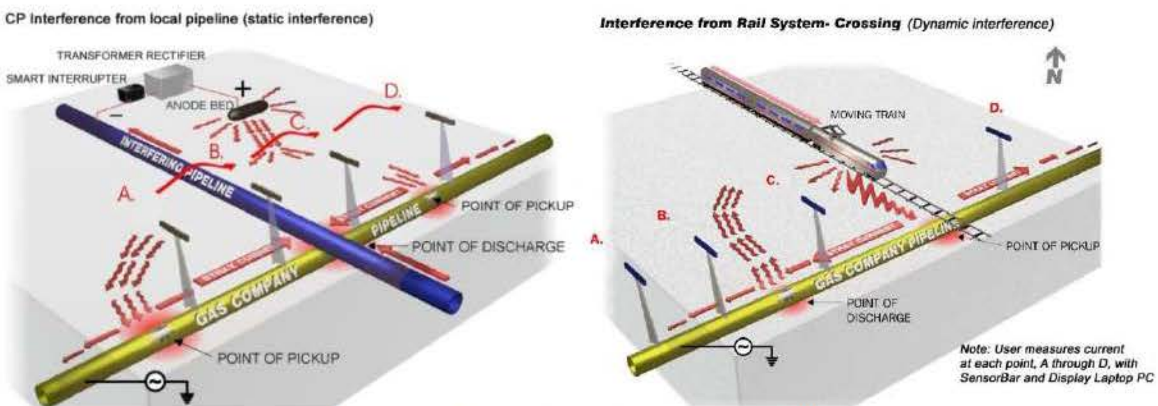


Figure 9 Examples of Stray Current[18]

[17] http://corrosion-doctors.org/StrayCurrent/Introduction.htm

[18] http://www.eastcomassoc.com/

LOG OF BORING B42 BY R.T. FRANKIAN

(2 PAGES)

BORING B-42

JOB NUMBER: 99-506-021
DATE DRILLED: 10/18/19
EQUIPMENT USED: 24" diameter Bucket Auger with heavy duty sampler
ELEVATION: 1426'
DRILLING CO.: Tri-Valley
LOGGED BY: MKM
BORING DEPTH: 0-47'

BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (LBS. PER CU. FT.)	N-VALUE	DEPTH (FEET)	SAMPLE LOCATION	GRAPHIC LOG	SOIL TYPE	DESCRIPTION
				0			ML	**TERRACE DEPOSITS (Qt)** SILT: trace clay, stiff, dry, light to medium brown
6	10	109	-	5			SM	SILTY SAND: very fine, very dense, dry, yellowish orange
4	19.6	109	-	10				
								trace clay
6	15.8	118	-	15				rocks
								yellowish brown
								@ 20': 1 FT. COBBLE LAYER N50W, 19SW: fine
8	8.1	96	-	20			GC	GRAVELLY SAND: fine to coarse, very dense, slightly moist, medium brown
7	6.5	128	-	25				
								SAUGUS FORMATION (TQs) PEBBLY SANDSTONE: fine to coarse, friable, moist, light brown
8	6.5	114	-	30				@ 30': BEDDING N28E, 19NW
11	7.6	116	-	35				
				40				

(CONTINUED ON THE FOLLOWING FIGURE)

Note: The log of subsurface conditions shown hereon is approximate and applies only at the specific location and date indicated. It is not warranted to be representative of subsurface conditions at other locations or times.

BOREHOLE LOG 99-506-021.GPJ FRANKIAN.GDT 12/18/19

LOG OF BORING

R.T. FRANKIAN & ASSOCIATES
RTF&A Job No. 99-506-021 1/30/2020

BORING B-42 (CONTINUED)

JOB NUMBER: 99-506-021
DATE DRILLED: 10/18/19
EQUIPMENT USED: 24" diameter Bucket Auger with heavy duty sampler
ELEVATION: 1426'
DRILLING CO.: Tri-Valley
LOGGED BY: MKM
BORING DEPTH: 0-47'

BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (LBS. PER CU. FT.)	N-VALUE	DEPTH (FEET)	SAMPLE LOCATION	GRAPHIC LOG	SOIL TYPE	DESCRIPTION
				45				
8	12.3	117	-					SANDSTONE: fine to medium with occasional coarse, soft, moist
								Bottom of Boring at 47 feet. No water. No caving. Boring backfilled and tamped.
				50				
				55				
				60				
				65				
				70				
				75				
				80				

Note: The log of subsurface conditions shown hereon is approximate and applies only at the specific location and date indicated. It is not warranted to be representative of subsurface conditions at other locations or times.

BOREHOLE LOG 99-506-021.GPJ FRANKIAN.GDT 12/18/19

LOG OF BORING

OIL WELL ABANDONMENT DOCUMENTS

(20 PAGES)

STATE OF CALIFORNIA-RESOURCES AGENCY

GEORGE DEUKMEJIAN, *Governor*

DEPARTMENT OF CONSERVATION
DIVISION OF OIL AND GAS
6401 TELEPHONE ROAD, SUITE 240
VENTURA, CALIFORNIA 93003-4458
(805) 654-4761

(1) Hold Processing Paperwork until well cut off

September 14, 1989

Larry Marr
Engineering Services Corp.
6017 Bristol Parkway
Culver City, CA. 90230

Dear Larry:

I have enclosed the following information that you have requested:

A) Well Summary Report indicating the location of the Jack L. Watkins "Legion" 1 well. Additionally is a plot map showing the location that was filed with well report.

B) Forms to be completed to obtain a permit to cut the well head or casing off in order to grade.

C) Well abandonment package. Included in this package is a list of abandonment contractors that perform the required well work. This will also give you instruction on the proper procedures to follow.

As per our telephone conversation, it is intended to not to build over or in proximity of this well. However, it is your intend to grade at this location at least 30 feet below the reported elevation for this well. If this is correct then the following instructions should be followed:

a) Submit two copies of a completed Supplementary Notice (Form OG123) I have partially filled in the form for you. You will be issued a Permit to Conduct Well Operations outlining the specify requirements .

b) Upon completion of the work, submit on Form (OG103) a history of the work.

Any questions, please feel free to contact me at (805) 654-47861.

Sincerely,

Steven A. Fields
Operations Engineer

attachment

FORM 165

STATE OF CALIFORNIA
DEPARTMENT OF NATURAL RESOURCES
DIVISION OF OIL AND GAS

REPORT OF CORRECTION OR CANCELLATION

Los Angeles California

June 24 1953

Mr Jack L Watkins
Route 1 Box 245-B
Newhall California

Dear Sir

In accordance with records (letter, form, etc.) dated February 27, 1953 the following change pertaining to your well No. "Legion" 1, Sec. 1, T. 3 N, R. 16 W, S B B. & M., Newhall field, District No. 1, is being made in our records:

☐ The corrected location is

☒ The corrected elevation is 1392 feet (ground)

☐ Report No. P152-1490, dated December 4, 1952, has been corrected ~~as follows:~~ as has also your notice to drill dated Novemver 24, 1952.

☐ Your notice to (Drill, abandon, etc.) dated , and our report No. P, issued in answer thereto, are hereby cancelled inasmuch as the work will not be done.

☐ Other:

Corrections Made as Follows:	By Whom
Form 114	
" 115	
" 121	
" 148	
Cards	
Production Reports	
Well Records (Folders	
(Reports	
Field Maps	No change
Map Book	No Change

Yours truly,

R. D. BUSH
State Oil and Gas Supervisor

By E H Musser
Deputy Supervisor ma

HRMA:my

cc - Messrs R D Bush (2)
Robin Willis

53460 11-51 5750 SPO

FORM 165

STATE OF CALIFORNIA
DEPARTMENT OF NATURAL RESOURCES
DIVISION OF OIL AND GAS

REPORT OF CORRECTION OR CANCELLATION

Los Angeles 15 California

June 22 1953

Mr Robin Willis
Route 1 Box 245-B
Newhall California

Engineer for Jack L Watkins

Dear Sir

In accordance with your well summary report (letter, form, etc.) dated February 27, 1953 the following change pertaining to your well No. "Legion" 1, Sec. 1, T. 3 N, R. 16 W, S B B. & M., Newhall field, District No. 1, is being made in our records:

☐ The corrected location is

☒ The corrected elevation is 1384' - ground
1392' - top of Kelly bushing

☐ Report No. , dated , has been corrected as follows:

☐ Your notice to drill (Drill, abandon, etc.) dated November 24, 1952, and our report No. P 152-1490, issued in answer thereto, a~~re hereby cancelled inasmuch as the work will not be done~~ have been corrected.

☐ Other:

EHMA:OH

Yours truly

R. D. Bush
State Oil and Gas Supervisor

By E. H. Musser
Deputy Supervisor

cc Messrs R D Bush (2)

Robin Willis Engineer
750 Subway Terminal Bldg
LOS ANGELES 13

70612 11-52 4150 SPO

FORM 159 (9-49)

STATE OF CALIFORNIA
DEPARTMENT OF NATURAL RESOURCES
DIVISION OF OIL AND GAS

REPORT OF WELL ABANDONMENT

Los Angeles 15, California, May 18, 1953

Mr Robin Willis Engineer
Jack L Watkins
Route 1 Box 245 B
Newhall California

Dear Sir

Your report of abandonment of Well No. "Legion" 1, Sec. 1, T. 3 N, R. 16 W, S B B. & M., Newhall oil field, Los Angeles County, dated February 27 1953, has been examined in conjunction with records filed in this office.

A review of the reports and records shows that the requirements of this Division, which are based on all information filed with it, have been fulfilled.

MAP	MAP BOOK	CARDS	BOND	FORMS 114	
18-A JFM	Jfm		5-18-53		EB

Yours truly

R. D. Bush
State Oil and Gas Supervisor

cc Mr R D Bush
Mr Jack L Watkins
Mr C J Threpp
Mr E B Wilson

es

By E. H. Musser
Deputy Supervisor

57615 2-52 12M SPO

ROBIN WILLIS
GEOLOGIST - PETROLEUM ENGINEER
750 SUBWAY TERMINAL BUILDING
LOS ANGELES 13, CALIFORNIA
MUTUAL 2156

DIVISION OF OIL AND GAS
RECEIVED
APR 9 1953
LOS ANGELES, CALIFORNIA

April 8, 1953

Division of Oil and Gas
1015 West Olympic Boulevard
Los Angeles (15), California

Attention: Mr. Murray-Aaron

Gentlemen:

Enclosed are two copies each of the completion reports of Jack L. Watkins' "Legion" 1 well in the Newhall Field as follows:

Well Summary Report,
History,
Core Descriptions, and
Sidewall Samples.

Two copies of the Schlumberger electrical log will be mailed to you by Mr. Jack L. Watkins or Schlumberger direct.

According to my records all the necessary reports, except these logs, have now been sent in on this well. Will you therefore kindly cancel the bond upon receipt of the Schlumberger electrical logs?

Yours very truly,

Robin Willis

GS

cc: Mr. Jack L. Watkins
Mr. N. Gordon Phillips

FORM 100.
CALIFORNIA STATE PRINTING OFFICE

SUBMIT LOG IN DUPLICATE

FILL THIS BLANK IN WITH TYPEWRITER. WRITE ON ONE SIDE OF PAPER ONLY

DIVISION OF OIL AND GAS
RECEIVED
APR 9 1953
LOS ANGELES, CALIFORNIA

STATE OF CALIFORNIA
DEPARTMENT OF NATURAL RESOURCES

DIVISION OF OIL AND GAS

WELL SUMMARY REPORT

Operator Jack L. Watkins Field Newhall

Well No. "Legion" 1 Sec. 1, T. 3-N, R. 16-W, S.B. B. & M.

Elevation above sea level 1392 K.B. feet.

Location* All depth measurements taken from top of Kelly bushing, which is 8 feet above ground.

In compliance with the provisions of Chapter 93, Statutes of 1939, the information given herewith is a complete and correct record of the present condition of the well and all work done thereon, so far as can be determined from all available records.

Date February 27, 1953. Signed Robin Willis

Robin Willis (Engineer or Geologist) J. L. Watkins (Superintendent) Title Robin Willis, Engineer.

Commenced drilling 11/26/52 Completed drilling 1/24/53 Drilling tools ~~Cable~~ Rotary

Total depth 3216' Plugged depth

GEOLOGICAL MARKERS	DEPTH

Junk

(*) From most easterly corner of Lot 4, Tract 2703, 150' N. 53° 43' W. along S. line of San Fernando Road, to the easterly corner of the leased property, thence 340' S. 27° 08' 53" W. along southeasterly property line to a point, thence 106' northwesterly at right angles. (Note: E. corner of Lot 4 is N. corner of Lot 5).

Commenced producing No production. (date) Flowing/gas lift/pumping (cross out unnecessary words)

	Clean Oil bbl. per day	Gravity Clean Oil	Per Cent Water including emulsion	Gas Mcf. per day	Tubing Pressure	Casing Pressure
Initial production						
Production after 30 days						

CASING RECORD (Present Hole)

Size of Casing (A. P. I.)	Depth of Shoe	Top of Casing	Weight of Casing	New or Second Hand	Seamless or Lapweld	Grade of Casing	Size of Hole Drilled	Number of Sacks of Cement	Depth of Cementing if through perforations
13-3/8"	24	0	used casing				18"	cemented solid.	

PERFORATIONS

Size of Casing	From	To	Size of Perforations	Number of Rows	Distance Between Centers	Method of Perforations
	ft.	ft.				
	ft.	ft.				
	ft.	ft.				
	ft.	ft.				
	ft.	ft.				

Electrical Log Depths 0 - 3159' (Attach Copy of Log)

FORM 103.
CALIFORNIA STATE PRINTING OFFICE

SUBMIT IN DUPLICATE

STATE OF CALIFORNIA
DEPARTMENT OF NATURAL RESOURCES

DIVISION OF OIL AND GAS

History of Oil or Gas Well

OPERATOR Jack L. Watkins FIELD Newhall

Well No. "Legion" 1, Sec. 1, T. 3-N, R. 16-W, S.B. B. & M.

Signed Robin Willis

Date February 27, 1953 Title Robin Willis, Engineer.

It is of the greatest importance to have a complete history of the well. **Use this form in reporting the history of all important operations at the well, together with the dates thereof, prior to the first production.** Include in your report such information as size of hole drilled to cementing or landing depth of casings, number of sacks of cement used in the plugging, number of sacks or number of feet of cement drilled out of casing, depth at which cement plugs started, and depth at which hard cement encountered. If the well was dynamited, give date, size, position and number of shots. If plugs or bridges were put in to test for water, state kind of material used, position and results of pumping or bailing.

Date	
1952	
11/26-12/13	Drilled 9-7/8" hole to 2956'.
12/13-16	Cored 7-5/8" hole, 2956-3159'.
12/16	Ran Schlumberger electric log. Put in cement plug, 30 sacks Construction Cement through 4-1/2" and 3-1/2" drill pipe hung at 3159'. Found top of cement at 3023', cleaned out to 3085'.
12/17	Ran Johnston formation tester. Set packer at 3000'. Plug would not hold weight, test failed. Drilled out cement plug.
12/17-18	Cored 7-5/8" hole, 3159-3216'.
12/20	Put in cement and sand plug, 50 sacks through 4-1/2" and 3-1/2" drill pipe hung at 3220'.
12/20-26	Shut down.
12/26	Cleaned out to 3085'.
12/27	Ran Johnston formation tester: tail at 3085', two sidewall packers, 3015 and 3005, 500' water cushion. Opened tool 8:43 A.M. Open 40 minutes, closed 1 hour with shut-in valve. Recovered cushion and 80' of gas-cut drilling fluid. Chart showed closed-in pressure build-up to 800#, indicated bottom hole pressure of 1000# plus.
12/27/52-1/23/53	Idle.
1/23/53	Cleaned out hole.
1/24	Put in cement plug, 30 sacks through 4-1/2" drill pipe hung at 3040' (filled to 2960'). Put in plug, 32 sacks through drill pipe hung at 40'. Plugging witnessed and approved by Division of Oil and Gas.
1/24/53	Abandoned well.

FORM 101
CALIFORNIA STATE PRINTING OFFICE

SUBMIT IN DUPLICATE

STATE OF CALIFORNIA
DEPARTMENT OF NATURAL RESOURCES

DIVISION OF OIL AND GAS

DIVISION OF OIL AND GAS
RECEIVED
APR 9 1953
LOS ANGELES, CALIFORNIA

LOG AND CORE RECORD OF OIL OR GAS WELL

Operator JACK L. WATKINS Field Newhall

Well No. "Legion" 1 Sec. 1, T. 3-N., R. 16-W., S.B. B. & M.

FORMATIONS PENETRATED BY WELL

DEPTH TO Top of Formation	DEPTH TO Bottom of Formation	Thickness	Drilled or Cored	Recovery	DESCRIPTION
					SIDEWALL SAMPLES
	2375'				Conglomerate, coarse, volcanic and quartz pebbles up to 1/2" in fine friable sand matrix, barren, light gray, no cut nor odor.
	2415'				Conglomerate as above. One 3/4" pebble.
	2484'				Gray sand, coarse, light gray, barren. One 1/4" pebble.
	2543'				Conglomerate, coarse with broken quartz pebble possibly 1" and matrix of coarse gray sand with inclusion of fine soft light greenish-brown oil sand with weak odor and pale straw cut.
	2633'				Gray sand, conglomeratic with pebbles up to 1/8", light gray, locally greenish, fine, looks oil stained but gives no cut.
	2664'				Conglomerate, one pebble 1-1/2" with coarse light gray, barren sand. No cut nor odor.
					- - 0 - -

FORM 101
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STATE OF CALIFORNIA
DEPARTMENT OF NATURAL RESOURCES

DIVISION OF OIL AND GAS

DIVISION OF OIL AND GAS
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APR 9 1953
LOS ANGELES, CALIFORNIA

LOG AND CORE RECORD OF OIL OR GAS WELL

Operator JACK L. WATKINS Field Newhall

Well No. "Legion" 1 Sec. 1, T. 3-N, R. 16-W, S.B. B. & M.

FORMATIONS PENETRATED BY WELL

DEPTH TO Top of Formation	DEPTH TO Bottom of Formation	Thickness	Drilled or Cored	Recovery	DESCRIPTION
2933'	2956'				CORE #1
		1.4'			Oil sand, soft, very fine medium light brown, good medium oil odor, dark brown cut. Sample.
		5.0			Shale, gray thinly bedded with abundant limy streaks and some fossils (fresh water clams?) and carbonaceous material. 21° average dip (17-30).
		1.1			Silty oil sand, very fine medium gray-brown, shaly streaks, fossils.
		4.2			Shale as above, locally very black, fossiliferous, carbonaceous, 15-22° dips.
		1.2			Shale, black, carbonaceous, 1/4" to 1/2" streaks fine oil sand.
		2.5			Shale as first above.
		.5			Shale, thin streaks oil sand.
		3.5			Shale, as first above, mostly chewed up in coring.
		.2			Shell, very hard, lightgray, limy shale.
		19.6'			
2986'	3006'				CORE #2
		4.0'			Oil silt, very firm, very fine, laminae, medium brown to blackish, clayey sand or silt locally dead dark gray, mostly medium dark brown with fair heavy oil odor, dark brown cut, looks too tight to produce.
		.6			Shell, same material but limy, hard light gray
		4.3			Oil silt as above.
		2.0			Shale, dark gray, barren very silty, fairly tough, bottom .5 limy.

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STATE OF CALIFORNIA
DEPARTMENT OF NATURAL RESOURCES

DIVISION OF OIL AND GAS

DIVISION OF OIL AND GAS
RECEIVED
APR 9 1953
LOS ANGELES, CALIFORNIA

LOG AND CORE RECORD OF OIL OR GAS WELL

Operator JACK L. WATKINS Field Newhall

Well No. "Legion" 1 Sec. 1, T. 3-N, R. 16-W, S.B. B. & M.

FORMATIONS PENETRATED BY WELL

DEPTH TO Top of Formation	DEPTH TO Bottom of Formation	Thickness	Drilled or Cored	Recovery	DESCRIPTION
					CORE #2 Cont'd.
		2.8'			Shale as last above with streaks oil silt, as above.
		2.4			Oil shell. Sandstone and sand, hard very fine down to medium hard coarse conglomeratic, pebbles up to 1/2". Sample at 1.4. Light amber cut, light brown to grayish mottled at bottom. Tight, probably not wet.
		16.1'			
3005'	3025'				CORE #3
		.8'			Shell, conglomeratic, hard limy sand, mottled light brown.
		1.0			Oil sand, fairly well saturated, conglomeratic fair light oil odor, sample at .5
		1.0			Shell as first above.
		1.6			Oil sand as above. Sample at 1.0
		1.0			Shell as first above.
		2.0			Oil sand, fine down to finely conglomeratic, fairly hard to hard, light brown, good odor, dark amber cut, sample at top.
		2.5			Shell as first above. One 65° shear with 45° slickensides.
		1.0			Gray sand, coarse conglomeratic, locally stained brown, medium hard to hard. Sample for porosity and permeability.
		.6			Shale, dark green, bentonitic.
		.7			Oil sand, fairly hard, shelly, fine, light brown, good medium oil odor.
		12.2'			

FORM 101
CALIFORNIA STATE PRINTING OFFICE

SUBMIT IN DUPLICATE

STATE OF CALIFORNIA
DEPARTMENT OF NATURAL RESOURCES

DIVISION OF OIL AND GAS

DIVISION OF OIL AND GAS
RECEIVED
APR 9 1953
LOS ANGELES, CALIFORNIA

LOG AND CORE RECORD OF OIL OR GAS WELL

Operator JACK L. WATKINS Field Newhall

Well No. "Legion" 1 Sec. 1, T. 3-N, R. 16-W, S.B. B. & M.

FORMATIONS PENETRATED BY WELL

DEPTH TO Top of Formation	DEPTH TO Bottom of Formation	Thickness	Drilled or Cored	Recovery	DESCRIPTION
					CORE #4
3025'	3050'	2.5'			Oil sand, coarse down to conglomeratic with 1/4" pebbles, very firm, easily friable, light brown, good medium oil odor, dark amber cut, sample at 1.3
		.3			Oil sand as above, shelly, fairly hard.
		1.0			Oil sand as first above. Sample at .4
		.5			Shell, very hard, limy light gray medium sand.
		1.3			Oil sand as first above. Sample at .8
		5.6			
					CORE #5
3050'	3074'	1.0'			Oil sand, fairly hard, very fine at top down to firm, easily friable, fine light brown, dark straw cut.
		1.7			Oil sand, very firm fine poorly sorted, light brown, free oil in partings, looks well saturated. Sample at 1.0'.
		5.5			Gray sand, very firm fairly easily friable, poorly sorted, fine to medium, looks rather "dirty" but as clean as oil sands, dead light gray, no cut, odor or fluorescence. Samples at .9 and 4.5', for porosity and permeability. .7' shelly hard sand at bottom.
		6.0			Oil sand, firm, easily friable, poorly sorted rather clayey, medium fine down to medium, light brown, good fresh kerosene or medium oil odor, dark reddish brown cut. Samples at 1.6, 3.7, 5.0,
		1.6			Conglomerate fairly soft, easily friable to very firm, pebbles up to 1" but mostly 1/8" to 1/4", mostly well saturated medium light brown with good medium oil odor and mahogany cut, but has streaks of gray barren material, clayey, high angle suggests faulting. Few slickensides at 45° in clay on one parting.
		15.8'			

FORM 101
CALIFORNIA STATE PRINTING OFFICE

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STATE OF CALIFORNIA
DEPARTMENT OF NATURAL RESOURCES

DIVISION OF OIL AND GAS

DIVISION OF OIL AND GAS
RECEIVED
APR 9 1953
LOS ANGELES, CALIFORNIA

LOG AND CORE RECORD OF OIL OR GAS WELL

Operator JACK L. WATKINS Field Newhall

Well No. "Legion" 1 Sec. 1, T. 3-N, R. 16-W, S.B. B. & M.

FORMATIONS PENETRATED BY WELL

DEPTH TO Top of Formation	DEPTH TO Bottom of Formation	Thickness	Drilled or Cored	Recovery	DESCRIPTION
3074'	3098'				CORE #6
		2.6'			Gray conglomerate very firm moderately friable pebbles up to 3". Mostly dead gray barren. Top .5 splotched with light brown oil stain.
		.9			Oil sand (?) fairly hard, coarse uniform, light brown speckled and shot through with gray. Looks poor. Sample at .7
		1.0			Gray conglomerate as first above, very little oil staining.
		2.5			Oil sand, firm, easily friable, finely conglomeratic, coarse, sharp, light brown, locally slightly splotched with gray. Looks fair. Good medium oil odor (not gassy). Two samples.
		7.0'			
3098'	3108'				CORE #7
		7.0			Siltstone fairly hard to hard, very fine, locally softer finely sandy, light to medium greenish gray to gray. 27° and 29° dips (washed-bedding).
3108'	3132'				CORE #8
		1.2'			Siltstone as above.
		1.5			Gray sand firm to very firm, very fine to fine light gray, no cut nor odor.
		5.5			Siltstone as above, two to three streaks very fine rather tight sand as above. No visible dips.
		8.2'			
3134'	3139' (corrected measurement).				CORE #9
		3.6'			Gray sand mostly fine to silty very firm to fairly hard locally medium fine, friable, dead gray, barren.

FORM 101
CALIFORNIA STATE PRINTING OFFICE

SUBMIT IN DUPLICATE

STATE OF CALIFORNIA
DEPARTMENT OF NATURAL RESOURCES

DIVISION OF OIL AND GAS

DIVISION OF OIL AND GAS
RECEIVED
APR 9 1953
LOS ANGELES, CALIFORNIA

LOG AND CORE RECORD OF OIL OR GAS WELL

Operator JACK L. WATKINS Field Newhall

Well No. "Legion" 1 Sec. 1, T. 3-N, R. 16-W, S.B. B. & M.

FORMATIONS PENETRATED BY WELL

DEPTH TO Top of Formation	DEPTH TO Bottom of Formation	Thickness	Drilled or Cored	Recovery	DESCRIPTION
					CORE #9 Cont'd.
		3.7'			Gray sand, very firm, very fine laminae with siltstone, dark fine lines to 1/2" bands, strong cross-bedding, best dips on shale streaks 22°-27°.
		4.3			Gray sand, fine down to coarse, pebbly on bottom, moderate to easily friable, barren.
		.9			Shale very sandy whorled fairly hard.
		.3			Oil sand soft, light brown little free oil, fair odor, fine fairly well sorted, amber cut.
		12.8'			
3159'	3177'				CORE #10
		7.0'			Bentonitic shale, bright green, slicked, little sand and hard pale green gray siltstone.
3177'	3200'				CORE #11
		1.7'			Green bentonite as above.
		3.4			Sandstone hard coarse pebbly pale green gray shelly. One 85° shear with vertical slickensides.
		.5			Gray sand, very firm, friable, light gray medium fine slightly clayey.
		5.6'			
3200'	3216'				CORE #12
		1.4			Sandstone, shelly green-gray, high angle shear with 50° slickensides.
		.8			Gray sand as in last core.
		2.2'			

FORM 109-D

STATE OF CALIFORNIA
DEPARTMENT OF NATURAL RESOURCES

DIVISION OF OIL AND GAS

Special Report on Operations Witnessed

No. T 153-157

Los Angeles 15
Calif. February 4 1953

Mr Robin Willis
XXX Route 1 Box 245-B
Newhall Calif.
Engineer XXXXX for JACK L WATKINS

DEAR SIR:

Operations at your well No. "Legion" 1 Sec. 1, T. 3 N, R. 16 W, S B B. & M., Newhall Field, in Los Angeles County, were witnessed by M. B. Albright, Inspector, representative of the supervisor, on January 24, 1953. There was also present D. B. Watkins, Operator; J. H. Jones, Driller.

Casing Record 13-3/8" cem. 24'. T.D. 3159', plugged with cement 3159'-3085', 3030'-2940'±, and 40' - to the surface.

Junk None

The operations were performed for the purpose of witnessing plugging operations in the process of abandonment.

The inspector arrived at the well at 11:00 a.m. and Mr. Watkins reported:

1. A 10-5/8" rotary hole was drilled from 24' to 2986'; a 7-5/8" rotary hole, 2986'-3159'.
2. Electrical log readings showed the top of the Braille zone at 3000'.
3. On January 20, 1953, 50 sacks of cement and 10 sacks of sand was pumped into the hole through 3-1/2" drill pipe hanging at 3159'.
4. Set cement and sand was drilled out of the hole from 3057'-3085'.
5. A test of the 3000'-3085' interval indicated that the sands would not be commercially productive.

THE INSPECTOR NOTED:

1. Thirty-five sacks of cement was pumped into the hole through 4-1/2" drill pipe hanging at 3030', calculated to fill to 2940'.
2. Thirty sacks of cement was pumped into the hole at 40', filling to the surface of the ground.

The operations were completed at 3:30 p.m.

THE PLUGGING OPERATIONS AS WITNESSED AND REPORTED ARE APPROVED.

MBA:OH

cc Mr Robin Willis Engineer
750 Subway Terminal Building
LOS ANGELES 13

W

R. D. BUSH
State Oil and Gas Supervisor
By E.H. Musser Deputy

44226 5-51 14,250 SPO

FORM 111 (1-49)

STATE OF CALIFORNIA
DEPARTMENT OF NATURAL RESOURCES

DIVISION OF OIL AND GAS

REPORT ON PROPOSED OPERATIONS

No. P 153-131

Los Angeles 15 Calif. January 27 1953

MR. Robin Willis
Route 1 Box 245-B
NEWHALL Calif.

124

Engineer ~~Agent~~ for JACK L. WATKINS

DEAR SIR:

Your proposal to abandon Well No. "Legion" 1,

Section 1, T. 3 N, R16 W, S B B. & M., Newhall Field, Los Angeles County,

dated Jan. 19 1953, received Jan. 20 1953, has been examined in conjunction with records filed in this office.

Present conditions as shown by the records and the proposal are as follows:

THE NOTICE STATES
"The present condition of the well is as follows:
1. Complete casing record.
13-3/8" Conductor pipe cemented at 24'.
10-5/8" hole drilled to 2780'. 7-5/8" hole drilled to 3159'. Cement plug from 3159' to 3085'.
There are no salt-water-sands above 3000'. There is a noncommercial oil zone 3000' to 3085'.
No production."

PROPOSAL
"The proposed work is as follows:
To put in 100' cement plug through 4-1/2" drill pipe hung at 3085'.
To put in 40' cement plug at surface, and abandon."

DECISION
THE PROPOSAL IS APPROVED PROVIDED THAT THIS DIVISION SHALL BE NOTIFIED TO WITNESS the placing of the proposed cement plugs.

EHMA:OH

cc Mr Robin Willis Engineer
750 Subway Terminal Bldg
LOS ANGELES 13

R. D. BUSH
State Oil and Gas Supervisor

By CHmusser Deputy

Bond No. 212336

FORM 108. 99470 11-48 14,250 SPO

STATE OF CALIFORNIA
DEPARTMENT OF NATURAL RESOURCES

DIVISION OF OIL AND GAS

DIVISION OF OIL AND GAS
RECEIVED
JAN 20 1953
LOS ANGELES, CALIFORNIA

Notice of Intention to Abandon Well

This notice must be given at least five days before work is to begin; one copy only

Los Angeles, Calif. January 19, 1953.

DIVISION OF OIL AND GAS
1015 West Olympic Boulevard
Los Angeles (15), Calif.

In compliance with Secs. 3228, 3229, 3230, 3231 and 3232, Ch. 93, Stat. 1939, notice is hereby given that it is our intention to abandon well No. "Legion" 1

Sec. 1, T. 3-N, R. 16-W, S.B. B. & M. Newhall Field,

Los Angeles County, commencing work on the 22nd day of January 1953.

The present condition of the well is as follows:

1. Complete casing record.

13-3/8" Conductor pipe cemented at 24'.
10-5/8" hole drilled to 2780',
7-5/8" hole drilled to 3159',
Cement plug from 3159' to 3085'.
There are no salt-water-sands above 3000'. There is a noncommercial oil zone from 3000' to 3085'.
No production.

2. Last produced. — (Date) — (Net oil) — (Gravity) — (Cut)

The proposed work is as follows:

To put in 100' cement plug through 4-1/2" drill pipe hung at 3085'.
To put in 40' cement plug at surface, and abandon.

abandon

MAP	MAP BOOK	CARDS	BOND	FORMS 114	121
		EB	212336	EB	EB

JACK L. WATKINS
(Name of Operator)
By Robin Willis
Robin Willis, Engineer.

ADDRESS NOTICE TO DIVISION OF OIL AND GAS IN DISTRICT WHERE WELL IS LOCATED

D

FORM 111 (1-49)

STATE OF CALIFORNIA
DEPARTMENT OF NATURAL RESOURCES

DIVISION OF OIL AND GAS

REPORT ON PROPOSED OPERATIONS

No. P 152-1490

Los Angeles 15 Calif. December 4 1952

MR. Robin Willis
Route 1, Box 245-B
NEWHALL Calif.

121

Engineer ~~Agent~~ for JACK L. WATKINS

DEAR SIR:

Your proposal to drill Well No. "Legion" 1,

Section 1, T. 3 N., R 16 W., S.B. B. & M., Newhall Field, Los Angeles County,

dated Nov. 24 1952, received Nov. 25 1952, has been examined in conjunction with records filed in this office.

Present conditions as shown by the records and the proposal are as follows:

THE NOTICE STATES

"The well is From most easterly corner of Lot 4, Tract 2703, 150' N. 53° 43' W. along S. line of San Fernando Road, to the easterly corner of the leased property, thence 340' S. 27° 08' 53" W. along southeasterly property line to a point, thence 106' Northwesterly at right angles. (Note: E. corner of Lot 4 is N. corner of Lot 5).

* Elevation of ground above sea level ~~1330 feet.~~ ~~1384 feet~~ 1392 feet.**
All depth measurements taken from top of Kelly bushing, which is 8 feet above ground.
We estimate that the first productive oil or gas sand should be encountered at a depth of about 2800 feet."

*Correction letter 6-22-53 OH
**Correction letter 6-24-53. my

PROPOSAL

"We propose to use the following strings of casing, either cementing or landing them as herein indicated:

Size of Casing	Weight	Grade and Type	Depth	Landed or Cemented
7"	23#	J-55 Seamless	2800	Cemented
5-1/2"	17#	J-55 Seamless	3000	Landed

Well is to be drilled with rotary tools.
It is understood that if changes in this plan become necessary we are to notify you before cementing or landing casing."

DECISION

THE PROPOSAL IS APPROVED PROVIDED THAT THIS DIVISION SHALL BE NOTIFIED TO WITNESS a test of the effectiveness of the 7" shut-off.

ERMA:OH

cc Mr Robin Willis Engineer
750 Subway Terminal Building
LOS ANGELES 13

MP

R. D. BUSH
State Oil and Gas Supervisor

By E. H. Messer Deputy

Bond No. 212336

O'DELL & COMPANY

INSURANCE

Thirty-four Fifty Wilshire Boulevard • *Los Angeles 5, California* • *Telephone Du-42119*

DIVISION OF OIL AND GAS
RECEIVED
NOV 26 1952
LOS ANGELES, CALIFORNIA

November 25, 1952

State of California
Division of Oil and Gas
1015 West Olympic Boulevard
Los Angeles, California

Gentlemen:

We are enclosing Oil Drilling Permit Bond #212336 of the Pacific Indemnity Company issued on behalf of Jack L. Watkins, in connection with the drilling of a well to be known as Legion No. 1, Section 1, Township 3 North, Range 16 West, S.B.B. & M., Los Angeles County, California.

Very truly yours,

R. W. O'Dell

R. W. O'Dell

RWO:CJ

FORM 105.
CALIFORNIA STATE PRINTING OFFICE

DIVISION OF OIL AND GAS
RECEIVED
NOV 25 1952
LOS ANGELES, CALIFORNIA

D
/3

STATE OF CALIFORNIA
DEPARTMENT OF NATURAL RESOURCES

037-12842

DIVISION OF OIL AND GAS

Notice of Intention to Drill New Well

This notice must be given and surety bond filed before drilling begins

Los Angeles, Calif. November 24, 1952 19

DIVISION OF OIL AND GAS
1015 West Olympic Boulevard
Los Angeles (15), Calif.

ma 18A 11-28-52 11-28-54 EB EB EB

MAP | FORMS 114 | 121

In compliance with Section 3203, Chapter 93, Statutes of 1939, notice is hereby given that it is our intention to commence the work of drilling well No. "Legion" 1, Sec. 1, T. 3-N
R. 16-W, S.B. B. & M., Newhall Field, Los Angeles County.

Legal description of lease 8 acres from Lot 4, Tract 2703, per attached plat.

The well is * feet N. or S., and feet E. or W. from
(Give location in distance from section corners or other corners of legal subdivision)

Elevation of ground above sea level ~~1330~~ ~~1384~~ ~~feet.~~ 1392 feet.**

All depth measurements taken from top of Kelly bushing, which is 8 feet above ground.

We estimate that the first productive oil or gas sand should be encountered at a depth of about 2800 feet.

We propose to use the following strings of casing, either cementing or landing them as herein indicated:

Size of Casing, Inches	Weight, Lb. Per Foot	Grade and Type	Depth	Landed or Cemented
7"	23#	J-55 Seamless	2800	Cemented
5-1/2"	17#	J-55 Seamless	3000	Landed

(*) From most easterly corner of Lot 4, Tract 2703, 150' N. 53° 43' W. along S. line of San Fernando Road, to the easterly corner of the leased property, thence 340' S. 27° 08' 53" W. along southeasterly property line to a point, thence 106' Northwesterly at right angles. (Note: E. corner of Lot 4 is N. corner of Lot 5). *

Well is to be drilled with rotary ~~cable~~ tools. * fr. NE Cor. Lot 4 Tract 2703, 320' SW'ly along E. line; thence 250' NW'ly at rt. angles.

It is understood that if changes in this plan become necessary we are to notify you before cementing or landing casing.

**Correction letter 6-24-53. my

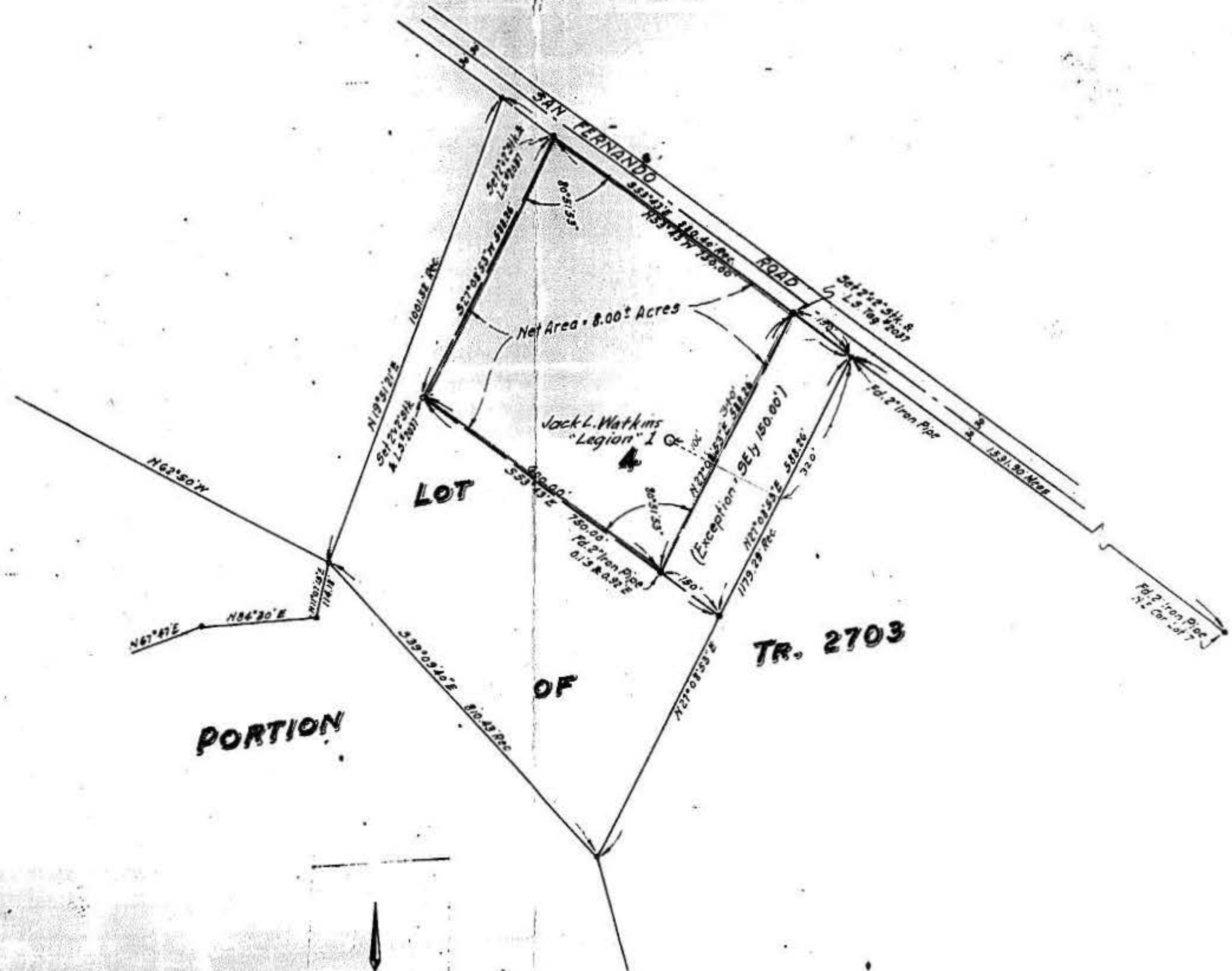
Address Route 1, Box 245-B
Newhall, California

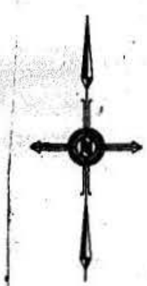
Telephone number Newhall 898

Jack L. Watkins
(Name of Operator)

By Robin Willis
Robin Willis, Engineer.

ADDRESS ONE COPY OF NOTICE TO DIVISION OF OIL AND GAS IN DISTRICT WHERE WELL IS LOCATED





9632-A	10-6-52	[illegible]
JOB No.	DATE	BY
REVISIONS		

MAP FOR

JACK WATKINS

SHOWING SURVEY of a PORTION of LOT 4,
TRACT No. 2703 as per M.B.28, Pg.20.

NEWHALL LOS ANGELES COUNTY CALIFORNIA.

SCALE 1" = 200' DATE OCTOBER 1952

JOB NO.		FILE NO.
9632-A	HOMER R. DULIN COMPANY LICENSED SURVEYORS 729 E. WILLOW ST. • PHONE 485-33 • LONG BEACH 6, CAL.	15-232