HARO, KASUNICH AND ASSOCIATES, INC.

CONSULTING GEOTECHNICAL & COASTAL ENGINEERS

Project No. SM10391.2
26 May 2018
Revised: 13 June 2018

MR. OWEN LAWLOR
Moss Beach Associates LLC
c/o Lawlor Land Use
612 Spring Street
Santa Cruz, CA 95060-2030

Subject: Geotechnical Investigation Update

Reference: Five Home Residential Development
Vallemar Street and Juliana Avenue
APN's 037-086-23, -26, -27, -28, & -29
Moss Beach, California

Dear Mr. Lawlor:

At your request, we are updating our 12 August 2016 (Project Number SM10391.2) Geotechnical Investigation Report with additional recommendations and current California Building Code (CBC 2016) standards. We have reviewed our prior project file and geotechnical recommendations for this project.

Project plans have changed since our 2016 Report. The current plans provided by Mesiti-Miller Engineering, Inc. (Job No. 15147, Revision Date 8/23/2017) show the project site split into 4 lots (Figure No. 2). Refer to our 2016 Report's Figure No. 2 for the original lot designations when reviewing the 2016 Report.

The project site is located within a seismically active area and strong seismic shaking is expected to occur over the lifetime of the project. Structures should be designed and constructed in accordance with the most current CBC (2016) and the recommendations of this report to minimize reaction to seismic shaking.

The results of our geotechnical investigation indicate the proposed residential structure on Lot 4 could be subject to ground settlement on the order of 3.0 inches total and 1.5 inches differential as a result of liquefaction. The proposed garage on Lot 4 could be subject to ground settlement on the order of 9.0 to 10.0 inches total and 4.0 to 5.0 inches differential as a result of liquefaction. Given the site conditions, the proposed structures on Lot 4 should be supported by a stiffened foundation system such as a structural mat slab with thickened edges or a grid foundation. The stiffened foundation will allow the proposed structures to float over ground effects that may occur during seismic activity including span voids. The foundations may require re-leveling after a design seismic event and should be evaluated by a licensed profession engineer with experience in foundation design after an earthquake. The stiffened foundation system should bear upon 24 inches of re-densified on-site soil in accordance with the recommendations of our August 2016 Report.

California Building Code (2016) Seismic Design Parameters

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

Longitude = -122.51675, Latitude = 37.53995

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

Site Class D

SM_S= 2.298 g

SM_1= 1.461 g

SD_S= 1.532 g

SD_1= 0.974 g

A maximum considered earthquake geometric mean (MCE_G) peak ground acceleration (PGA) was estimated using the Figure 22-7 of the ASCE Standard 7-10. The mapped PGA was 0.896 g and the site coefficient F_{PGA} for Site Class D is 1.0. The MCE_G peak ground acceleration adjusted for Site Class effects is PGA_M = FPGA * PGA

$$\boxed{PGA_M = 1.0 * 0.896\text{ g} = 0.896\text{ g}}$$

Based on these considerations, the risk of substantial structural damage from earthquakes appears relatively low for well-built structures which incorporate lateral shear bracing and current California Building Code (CBC) requirements into their design and construction. These considerations will be the primary factors in reducing the potential for earthquake damage to the project in the future.

<u>Geological Hazards</u>

Liquefaction

During an earthquake, seismic waves travel through the earth and vibrate the ground. In cohesionless, granular material having low relative density (loose to medium dense sands for example), this vibration can disturb the particle framework leading to increased compaction of the material and reduction of pore space between the framework grains. If the sediment is saturated, water occupying the pore spaces resists this compaction and exerts pore pressure that reduces the contact stress between the sediment grains. With continued shaking, transfer of intergranular stress to pore water can generate pore pressures great enough to cause the sediment to lose its strength and change from a solid state to a liquefied state. This mechanical transformation termed liquefaction can cause various kinds of ground failure at or near the ground surface. The liquefaction process typically occurs at depths less than 50 feet below the ground surface. Liquefaction can occur at deeper intervals, given the right conditions, however ground manifestations have been found to be relatively minor.

Based on the presence of groundwater in our test borings B-3 and B-4, there is a moderate potential for liquefaction to occur at Lot 4 of the project site. A model was developed using our lab results and subsurface information collected from our test borings B-3 and B-4. The model was created using Liquefy Pro software to quantify potential for liquefaction, dynamic compaction, and related ground effects within the upper 33 to 39 feet below ground surface (bgs).

The coarse grained soil layers comprised of clayey sand or sand with silt were assumed to have potential for dynamic compaction and/or liquefaction where below the water table. The fine grained soils comprised of sandy clay are non-liquefiable and assigned that way in our model. The soil encountered in Test Boring 3 and Test Boring 4 were visually classified in the field following ASTM D2488 “Description and Identification of Soils Visual Manual Procedure”. For use in our model the soil visually classified as sand with silt was assume to have a fines content of 5 percent and soil visually classified as clayey sand was assumed to have a fines content of 12 percent. This is on the low end of the fines content for both soil types so is considered conservative for use in the liquefaction analysis. The results of our analysis indicate there is potential for the sand with silt and or clayey sand layers to liquefy between approximately 17 to 33 feet bgs during ground shaking from a design seismic event.

The proposed residential structure on Lot 4 could be subject to ground settlement on the order of 3.0 inches total and 1.5 inches differential as a result of liquefaction. The proposed garage on Lot 4 could be subject to ground settlement on the order of 9.0 to 10.0 inches total and 4.0 to 5.0 inches differential as a result of liquefaction. Given the site conditions, the proposed structures on Lot 4 should be supported by a stiffened foundation system such as a structural mat slab with thickened edges or a grid foundation. The stiffened foundation will allow the proposed structures to float over ground effects that may occur during seismic activity as well as span voids. The foundations may require re-leveling after a design seismic event and should be evaluated by a licensed profession engineer with experience in foundation design after an earthquake. The stiffened foundation systems (slab or grid) and its thickened edges should bear upon a minimum 24 inches of re-densified on-site soils in accordance with the recommendations of our August 2016 Report.

Building Codes and Site Class

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

-2016 California Building Code (CBC); and

-2016 Green Building Standards Code (CAL Green)

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

Soil Properties

Based on our field exploration and results of laboratory tests the soils encountered were simplified into two soil types. Soil Type 1: Clay Soil Coastal Terrace and Soil Type 2: Silt Sand Clay Mixture Terrace Deposit, Soil type 3: Bedrock Formation. The geotechnical strength parameters of the soil types are summarized in the table below.

Table 1: Geotechnical Design Values

Soil Stratum	γ_t (lbs/ft^3)	$^\circ$(degrees)	Cohesion (lbs/ft^2)
Soil 1	123	10	1000
Soil 2	113	43	0
Soil 3	135	45	1000

Quantitative Slope Stability Analysis

Stability analysis was performed on the worst case or critical cross section cut through the coastal bluff and Lot 4. The critical section (Cross Section 3) was selected by HKA and developed using a topographical map prepared by Gary Ifland surveyor, Inc. A copy of the slope stability cross section is included with this letter. The slope stability analysis was performed to quantify the potential for bluff failure that could impact the proposed building site. It also corroborated the development of the recommended 50-year future coastal bluff recession slope stability setback line.

General Methodology

Slope failures or landslides can cause problems including encroachment and undermining of engineered structures. Failures of slopes occur when stress acting on the soil mass is greater than its internal strength (shear strength). A slope is considered stable when the strength of its soil mass is greater than the stress field acting within it. Some common variables influencing stress are gravity (steeper slopes), hydrostatic pressure (perched groundwater), bearing pressures (proposed structures), and seismic surcharge (earthquake shaking).

Various methods of analyzing stability of slopes yield a factor of safety. A factor of safety is determined by dividing the resisting forces within the slope soils by the driving forces within the slope (stress field). A factor of safety (FS) greater than or equal to 1.0 is considered to be in equilibrium. A FS less than 1.0 is a potentially un-stable slope condition. HKA considers the potential for instability of a slope or hillside with a FS against sliding greater than or equal to 1.10 under seismic loading conditions and 1.50 under static loading conditions to be low.

Quantitative Analysis with GSTABL7

The analysis was completed with the aid of GSTABL7 software. A model for the section was defined with the input parameters consisting of slope geometry, soil properties, loading conditions, and pore water pressure ratio. Each model was evaluated under static and seismic loading. The analysis calculates the factor of safety against sliding for the failure surface(s).

A critical surface was selected for this model based on the failure surface in the Coastal Bluff Recession Cross Section 3. GSTABL7 program uses the Janbu Method to determine normal and resistive forces in each slice. The forces in each slice are then summed up for total force acting on the mass.

Seismic Coefficient

The ground motion parameter used in pseudo static analysis is referred to as the seismic coefficient “k”. The selection of a seismic coefficient has relied heavily on engineering judgment, local building code, and professional publications. Current version of the California Building Code contains reference to maps of peak ground acceleration (PGA) based on site latitude and longitude. For this project the mapped PGA is 0.896g. The PGA is multiplied by a factor related to the seismicity of the site to obtain the seismic coefficient. The factor was estimated to be 0.58 by using Figure 1 of Chapter 5 Analysis of Earthquake-Induced Landslide Hazards in CGS *Special*

Publication 117 Guidelines For Analyzing and Mitigating Seismic Hazards in California 2008.

The multiplying factor was developed as part of a screen analysis procedure for seismic slope stability by Stewart, Blake, and Hollingsworth. The multiplier results in a percentage of the peak which represents the more repeatable ground motion. The assumption is the site can tolerate at least 2 inches of displacement during a design seismic event. The higher the multiplying factor the less displacement during a design seismic event is assumed to be tolerable by site improvements. For example if the full peak ground acceleration is used in the analysis (multiplier of 1.0) it is assumed 0 inches of displacement is tolerated during a design seismic event. For this project we assumed 2 inches of ground displacement is tolerable resulting a horizontal seismic coefficient of 0.513g.

<u>Geometric Assumptions</u>
For our analysis, the failure surface was focused within Soil 1 “Unclassified Fill and Coastal Terrace Deposits” due to its vulnerability to bluff failures relative to Soil 2 “Bedrock Formation” which is much more resistance to erosional processes and slope failures.

<u>Slope Stability Conclusions</u>
The computed factors of safety for the trial failure surfaces are greater than 1.50 under static loading conditions and 1.10 for pseudo-static conditions. The results of our analysis indicate that the portion of the coastal bluff comprised of terrace deposits is stable at slope gradients of 1.5:1 (H:V) or flatter. Based on these results the potential for instability of the coastal bluff impacting the proposed home sites is low. However, portions of the coastal bluff that have slope gradients steeper than 1.5:1 (H:V) are predicted to have bluff failures until the slope gradient recesses to 1.5:1 (H:V). A portion of the coastal terrace deposit portion of the bluff along Cross Section 1 and 4 are flatter than 1.5:1 (H:V). These slope gradients are estimated to be stable and therefore slope stability analysis was not performed on these cross sections. Section 2 is similar to Section 3 but a little flatter and qualitatively would have a higher factor of safety against sliding compared to Section 3. The results of the slope stability analyses are summarized in the following table as well as presented graphically in Appendix of this letter.

Table 2: Slope Stability Analysis Results

Bluff Recession Section	Loading Condition	Minimum Factor of Safety Against Sliding	Meet or Exceed Required FS
3	Static	2.26	Yes
3	Pseudo Static	1.14	Yes

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

Additional Geotechnical Recommendations

Based on review of our prior Geotechnical Investigation and associated addendums, we present the following additional recommendations to be used as guidelines for preparing project plans and specifications. All recommendations from Haro, Kasunich & Associates' (HKA) 12 August 2016 Geotechnical Investigation Report should be followed as well. Lots 1, 2 &3 should have foundations designed per 2016 Report.

Stiffened Foundations for Lot 4

1. Based on the site and soil characteristics, the proposed residential structures should be supported by a stiffened foundation system such as a structural slab with thickened edges or a grid foundation. The stiffened foundation system including thickened edges should bear upon a minimum 24 inches of engineered fill, moisture conditioned, and compacted in accordance with this report. Refer to the *Site Grading* section.

2. The structural slab should be a minimum 10 inches thick with minimum 12 inch thick edges embedded a minimum 8 inches below ground surface. If a grid foundation is selected it should be a minimum 24 inches thick and 18 inches wide with a minimum two (2) – No. 6 steel reinforcement placed top and bottom. Actual dimensions of the foundation and reinforcing should be provided by the

project structural designer based on the actual loads transmitted to the foundation.

3. Stiffened foundations constructed to the given criteria may be designed for the following allowable bearing capacities:
    a) *Grid Foundation:* 1,200 psf for dead plus live loads for 24 inch deep footing depth.
    b) *Structural Mat Slab Foundation:* refer to the appendix for bearing capacity graph and soil reaction modulus graph. Graphs assume 28'x28' garage slab and 45'x65' house slab.
    c) A one-third increase for seismic loading
    d) Coefficient of friction of 0.30

4. Foundation excavations should be thoroughly cleaned, moisture conditioned and observed by the HKA or representative prior to placing forms and steel. Observation of foundation excavations allows anticipated soil conditions to be correlated to those inferred from our investigation and to verify the foundation excavations are in accordance with our recommendations.

5. Provided our recommendations are incorporated into the design and construction of the project, the structural slab or grid foundation supporting the garage structure should be designed to tolerate on the order of 9.0 to 10.0 inches of total settlement and 4.0 to 5.0 inches of differential settlement. The structural slab or grid foundation system supporting the house on lot 4 should be designed to tolerate on the order of 3.0 inches of total settlement and 1.5 inch of differential settlement. Both garage and house foundation should be designed to span an unsupported length of 12 feet in diameter in any location within the interior and cantilever a distance of 5 feet along the edges. This evaluation for areas of non-support is purely empirical and not intended to model actual site performance. The purpose is to establish foundation stiffness to control differential movement.

Plan Review, Construction Observation and Testing

The above recommendations and our 12 August 2016 Geotechnical Investigation Report should be used as guidelines for preparing project plans and specifications. **Haro, Kasunich & Associates** should be commissioned to review project grading and foundation plans before construction and to observe, test and advise during earthwork and foundation construction. This additional opportunity to examine the site will allow us to compare subsurface conditions exposed during construction with those inferred from this investigation. Unusual or unforeseen soil conditions may require supplemental evaluation by the geotechnical engineer.

Should you have any questions concerning this letter report, please call our office.

Respectfully Submitted,

HARO, KASUNICH & ASSOCIATES, INC.

Reviewed by:

REGISTERED PROFESSIONAL ENGINEER
MOSES E. CUPRILL
No. 78904
Exp. 3/20
CIVIL
STATE OF CALIFORNIA

Moses Cuprill, P.E.
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LIMITATIONS AND UNIFORMITY OF CONDITIONS

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

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

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

APPENDIX A

Site Vicinity Map

Boring Site Plan

Slope Stability Results

Liquefaction Results

Allowable Bearing Capacity for Mat Foundations

Soil Reaction Modulus for Mat Foundations

SITE LOCATION

SITE VICINITY MAP
VALLEMAR STREET & JULIANA AVENUE
MOSS BEACH, CALIFORNIA
MOSS BEACH ASSOCIATES

SCALE:	NTS
DRAWN BY:	BRS
DATE:	MAY 2018
REVISED:	
JOB NO.	SM10391.2

HARO, KASUNICH & ASSOCIATES, INC.
GEOTECHNICAL AND COASTAL ENGINEERS
116 E. LAKE AVENUE, WATSONVILLE, CA 95076
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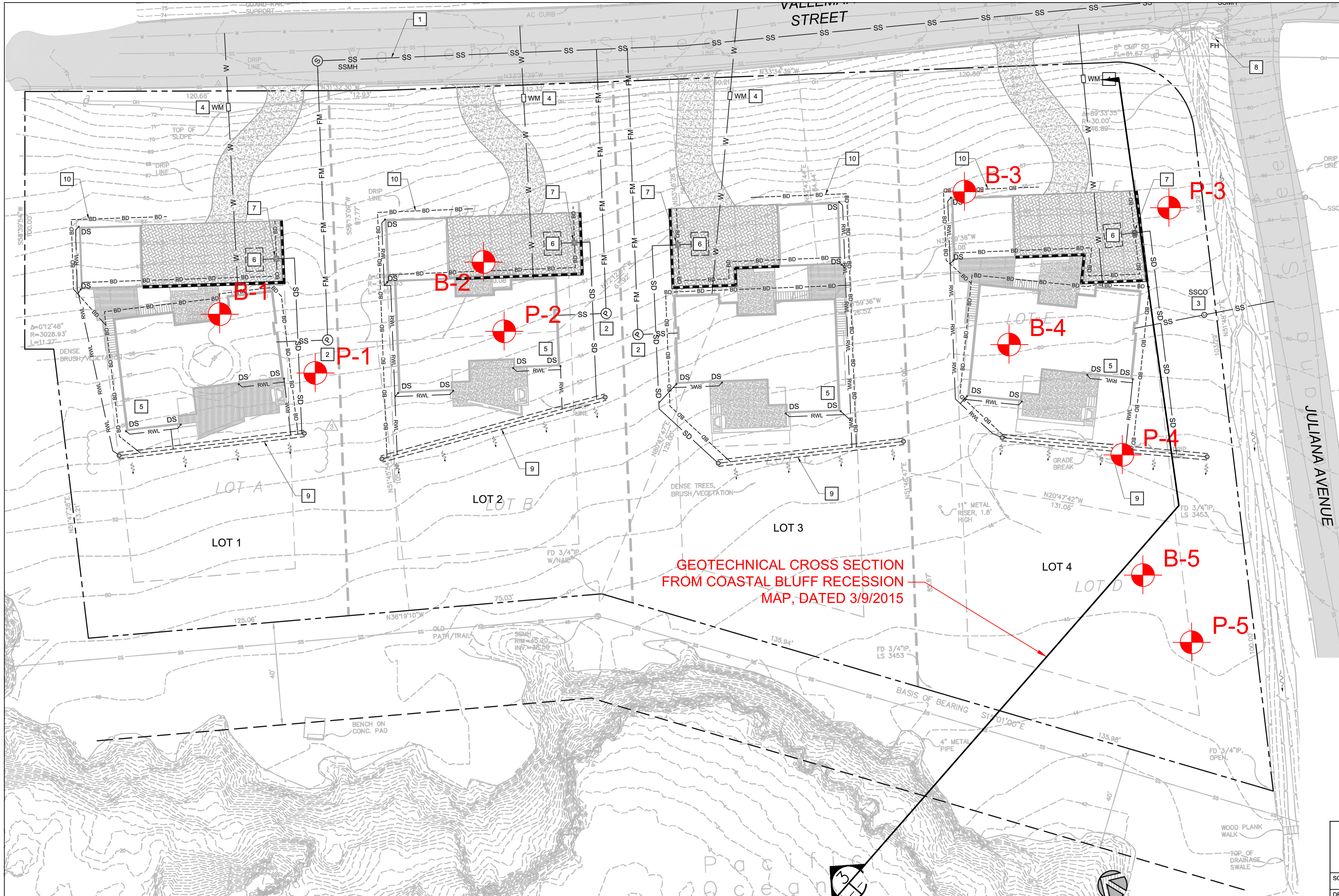
FIGURE NO. 1

SHEET NO.

FROM:
USGS 20 ft. contour interval



NOTES:

1. MAP FROM MESITI-MILLER ENGINEERING, INC., JOB NO. 15147, REVISION DATE 8/23/2017.

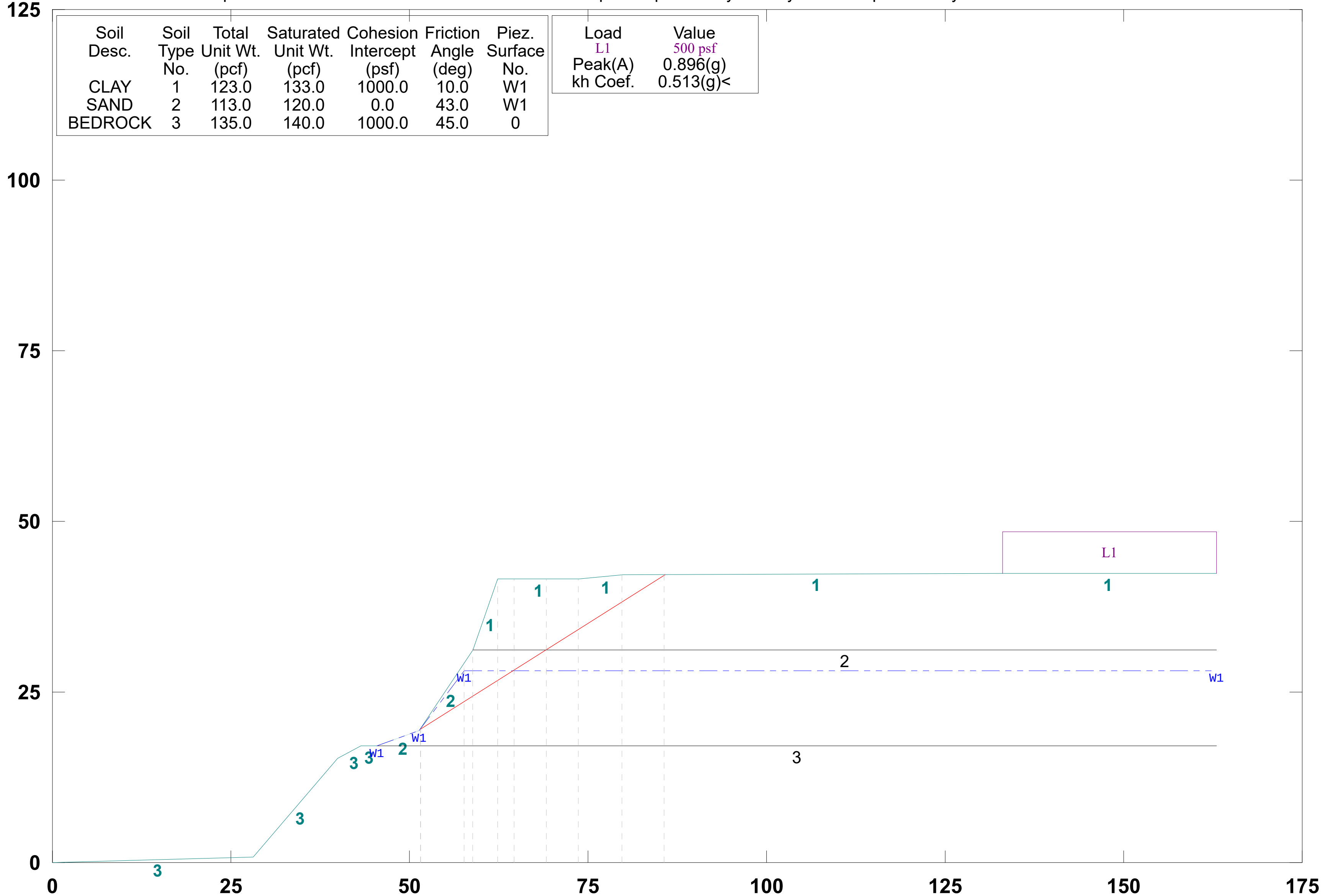
KEY: B-1 = SOIL BORING LOCATION

KEY: P-1 = PERCOLATION TEST LOCATION

BORING SITE PLAN VALLEMAR STREET & JULIANA AVENUE MOSS BEACH, CALIFORNIA MOSS BEACH ASSOCIATES	
SCALE: 1" = 30'	
DRAWN BY: BRS	HARO, KASUNICH & ASSOCIATES, INC. GEOTECHNICAL AND COASTAL ENGINEERS 116 E. LAKE AVENUE, WATSONVILLE, CA 95076 (831) 722-4175
DATE: MAY 2018	
REVISED:	
JOB NO. SM10391.2	
FIGURE NO. 2	SHEET NO.

VALLEMAR ST AND JULIANA AVE SLOPE ST ABILITY ANALYSIS SECTION 3

c:\users\hka\dropbox\hka\10391.2 vallemar street - moss beach\report\slope stability\stability section 3.plt Run By: Brian Shedden 6/13/2018 07:25PM



GSTABL7 v.2 FSmin=1.14

Factor Of Safety Is Calculated By The Simplified Janbu Method

***** GSTABL7 *****

** GSTABL7 by Dr. Garry H. Gregory, Ph.D.,P.E.,D.GE **
** Original Version 1.0, January 1996; Current Ver. 2.005.2, Jan. 2011 **

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SLOPE STABILITY ANALYSIS SYSTEM

Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.

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Run By: Brian Shedden
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Output Filename: C:\Users\HKA\Dropbox\HKA\10391.2 Vallemar Street - Moss Beach\REPORT\Slope Stability\stability section 3.OUT
Unit System: English
Plotted Output Filename: C:\Users\HKA\Dropbox\HKA\10391.2 Vallemar Street - Moss Beach\REPORT\Slope Stability\stability section 3.PLT
PROBLEM DESCRIPTION: VALLEMAR ST AND JULIANA AVE SLOPE STABILITY ANALYSIS SECTION 3

BOUNDARY COORDINATES
11 Top Boundaries
13 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	0.00	28.20	0.80	3
2	28.20	0.80	40.00	15.20	3
3	40.00	15.20	43.20	17.10	3
4	43.20	17.10	45.40	17.20	3
5	45.40	17.20	51.40	19.30	2
6	51.40	19.30	58.80	31.20	2
7	58.80	31.20	62.30	41.50	1
8	62.30	41.50	73.70	41.50	1
9	73.70	41.50	79.80	42.20	1
10	79.80	42.20	132.70	42.30	1
11	132.70	42.30	163.00	42.30	1
12	58.80	31.20	163.00	31.20	2
13	45.40	17.20	163.00	17.20	3

Default Y-Origin = 0.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS
3 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	123.0	133.0	1000.0	10.0	0.00	0.0	1
2	113.0	120.0	0.0	43.0	0.00	0.0	1
3	135.0	140.0	1000.0	45.0	0.00	0.0	0

1 PIEZOMETRIC SURFACE(S) SPECIFIED
Unit Weight of Water = 62.40 (pcf)
Piezometric Surface No. 1 Specified by 4 Coordinate Points
Pore Pressure Inclination Factor = 0.50

Point No.	X-Water (ft)	Y-Water (ft)
1	45.40	17.20
2	51.40	19.30
3	57.60	28.20
4	163.00	28.20

BOUNDARY LOAD(S)
1 Load(s) Specified

Load No.	X-Left (ft)	X-Right (ft)	Intensity (psf)	Deflection (deg)
1	133.00	163.00	500.0	0.0

NOTE - Intensity Is Specified As A Uniformly Distributed

Force Acting On A Horizontally Projected Surface.
Specified Peak Ground Acceleration Coefficient (A) = 0.896(g)
Specified Horizontal Earthquake Coefficient (kh) = 0.513(g)
Specified Vertical Earthquake Coefficient (kv) = 0.000(g)
Specified Seismic Pore-Pressure Factor = 0.000
Janbu's Empirical Coef. is being used for the case of c & phi both > 0
Trial Failure Surface Specified By 2 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	51.500	19.461
2	85.720	42.211

Janbu's Empirical Coefficient (fo) = 1.000
* * Factor Of Safety Is Calculated By The Simplified Janbu Method * *
Factor Of Safety For The Preceding Specified Surface = 1.142
Table 1 - Individual Data on the 9 Slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2	6.1	2049.2	0.0	707.5	0.0	0.0	1051.2	0.0	0.0
3	1.2	893.0	0.0	385.3	0.0	0.0	458.1	0.0	0.0
4	3.5	4547.0	0.0	714.0	0.0	0.0	2332.6	0.0	0.0
5	2.3	3985.3	0.0	137.0	0.0	0.0	2044.5	0.0	0.0
6	4.5	6481.7	0.0	0.0	0.0	0.0	3325.1	0.0	0.0
7	4.5	4911.2	0.0	0.0	0.0	0.0	2519.5	0.0	0.0
8	6.1	4203.4	0.0	0.0	0.0	0.0	2156.4	0.0	0.0
9	5.9	1428.9	0.0	0.0	0.0	0.0	733.0	0.0	0.0

Table 2 - Base Stress Data on the 9 Slices

Slice No. *	Alpha (deg)	X-Coord. Slice Cntr (ft)	Base Leng. (ft)	Available Shear Strength (psf)	Mobilized Shear Stress (psf)
1	33.62	51.51	0.03	0.87	1.17
2	33.62	54.56	7.30	174.37	330.71
3	33.62	58.20	1.44	346.06	729.90
4	33.62	60.55	4.20	819.65	1274.26
5	33.62	63.47	2.82	1198.20	1666.89
6	33.62	66.90	5.42	1042.58	1408.88
7	33.62	71.43	5.45	1296.68	1060.46
8	33.62	76.75	7.33	1221.38	675.88
9	33.62	82.76	7.11	1135.41	236.74

Sum of the Resisting Forces (including Pier/Pile, Tieback, Reinforcing
Soil Nail, and Applied Forces if applicable) = 38340.75 (lbs)
Average Available Shear Strength (including Tieback, Pier/Pile, Reinforcing,
Soil Nail, and Applied Forces if applicable) = 933.04(psf)
Sum of the Driving Forces = 33567.70 (lbs)
Average Mobilized Shear Stress = 816.88(psf)
Total length of the failure surface = 41.09(ft)
*** SEISMIC SLOPE DISPLACEMENT DATA ***
(Note: kv is set = zero for displacement calculations)
Seismic Yield Coefficient (ky) = 0.6451(g)
Calculated Newmark Seismic Displacement = 0.168(ft)
Non-Symmetrical Sliding Resistance Has Been Specified
for Downhill Sliding.
**** END OF GSTABL7 OUTPUT ****

VALLEMAR ST AND JULIANA AVE SLOPE ST ABILITY ANALYSIS SECTION 3

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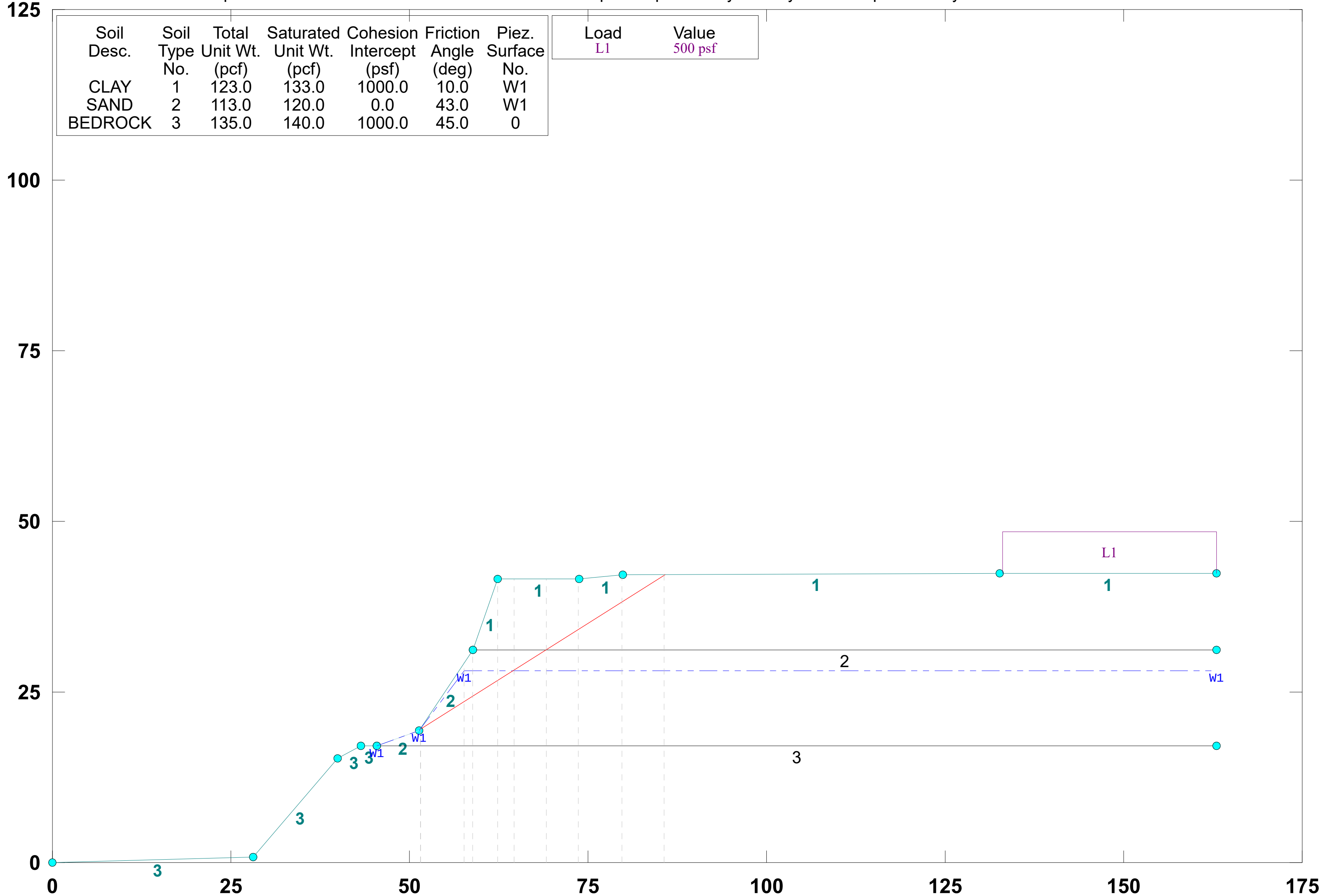

GSTABL7 v.2 FSmin=2.24

Factor Of Safety Is Calculated By The Simplified Janbu Method

***** GSTABL7 *****

** GSTABL7 by Dr. Garry H. Gregory, Ph.D.,P.E.,D.GE **
** Original Version 1.0, January 1996; Current Ver. 2.005.2, Jan. 2011 **

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SLOPE STABILITY ANALYSIS SYSTEM
Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
****

Analysis Run Date: 5/26/2018
Time of Run: 11:26AM
Run By: Brian Shedden
Input Data Filename: C:\Users\HKA\Dropbox\HKA\10391.2 Vallemar Street - Moss Beach\REPORT\Slope Stability\stability section 3.in
Output Filename: C:\Users\HKA\Dropbox\HKA\10391.2 Vallemar Street - Moss Beach\REPORT\Slope Stability\stability section 3.OUT
Unit System: English
Plotted Output Filename: C:\Users\HKA\Dropbox\HKA\10391.2 Vallemar Street - Moss Beach\REPORT\Slope Stability\stability section 3.PLT
PROBLEM DESCRIPTION: VALLEMAR ST AND JULIANA AVE SLOPE STABILITY ANALYSIS SECTION 3

BOUNDARY COORDINATES
11 Top Boundaries
13 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	0.00	28.20	0.80	3
2	28.20	0.80	40.00	15.20	3
3	40.00	15.20	43.20	17.10	3
4	43.20	17.10	45.40	17.20	3
5	45.40	17.20	51.40	19.30	2
6	51.40	19.30	58.80	31.20	2
7	58.80	31.20	62.30	41.50	1
8	62.30	41.50	73.70	41.50	1
9	73.70	41.50	79.80	42.20	1
10	79.80	42.20	132.70	42.30	1
11	132.70	42.30	163.00	42.30	1
12	58.80	31.20	163.00	31.20	2
13	45.40	17.20	163.00	17.20	3

Default Y-Origin = 0.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS
3 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	123.0	133.0	1000.0	10.0	0.00	0.0	1
2	113.0	120.0	0.0	43.0	0.00	0.0	1
3	135.0	140.0	1000.0	45.0	0.00	0.0	0

1 PIEZOMETRIC SURFACE(S) SPECIFIED
Unit Weight of Water = 62.40 (pcf)
Piezometric Surface No. 1 Specified by 4 Coordinate Points
Pore Pressure Inclination Factor = 0.50

Point No.	X-Water (ft)	Y-Water (ft)
1	45.40	17.20
2	51.40	19.30
3	57.60	28.20
4	163.00	28.20

BOUNDARY LOAD(S)
1 Load(s) Specified

Load No.	X-Left (ft)	X-Right (ft)	Intensity (psf)	Deflection (deg)
1	133.00	163.00	500.0	0.0

NOTE - Intensity Is Specified As A Uniformly Distributed

```
       Force Acting On A Horizontally Projected Surface.
Specified Peak Ground Acceleration Coefficient (A) =   0.890(g)
Specified Horizontal Earthquake Coefficient (kh) =   0.510(g)
Specified Vertical Earthquake Coefficient (kv) =   0.000(g)
Specified Seismic Pore-Pressure Factor =   0.000
EARTHQUAKE DATA HAS BEEN SUPPRESSED
Janbu's Empirical Coef. is being used for the case of  c & phi both > 0
Trial Failure Surface Specified By  2 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1        51.500      19.461
    2        85.720      42.211
Janbu's Empirical Coefficient (fo) =  1.000
* * Factor Of Safety Is Calculated By The Simplified Janbu Method * *
Factor Of Safety For The Preceding Specified Surface =  2.240
     ***Table 1 - Individual Data on the    9 Slices***
```

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2	6.1	2049.2	0.0	707.5	0.0	0.0	0.0	0.0	0.0
3	1.2	893.0	0.0	385.3	0.0	0.0	0.0	0.0	0.0
4	3.5	4547.0	0.0	714.0	0.0	0.0	0.0	0.0	0.0
5	2.3	3985.3	0.0	137.0	0.0	0.0	0.0	0.0	0.0
6	4.5	6481.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0
7	4.5	4911.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0
8	6.1	4203.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0
9	5.9	1428.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0

```
     ***Table 2 - Base Stress Data on the    9 Slices***
```

Slice No. *	Alpha (deg)	X-Coord. Slice Cntr (ft)	Base Leng. (ft)	Available Shear Strength (psf)	Mobilized Shear Stress (psf)
1	33.62	51.51	0.03	1.05	0.66
2	33.62	54.56	7.30	210.70	186.67
3	33.62	58.20	1.44	418.17	411.99
4	33.62	60.55	4.20	990.45	719.26
5	33.62	63.47	2.82	1447.87	940.88
6	33.62	66.90	5.42	1259.83	795.25
7	33.62	71.43	5.45	1358.66	598.58
8	33.62	76.75	7.33	1279.77	381.50
9	33.62	82.76	7.11	1189.68	133.63

```
 Sum of the Resisting Forces (including Pier/Pile, Tieback, Reinforcing
 Soil Nail, and Applied Forces if applicable) =   42446.45 (lbs)
 Average Available Shear Strength (including Tieback, Pier/Pile, Reinforcing,
 Soil Nail, and Applied Forces if applicable) =   1032.95(psf)
 Sum of the Driving Forces =    18947.37 (lbs)
 Average Mobilized Shear Stress =     461.09(psf)
 Total length of the failure surface =     41.09(ft)
               **** END OF GSTABL7 OUTPUT ****
```

LIQUEFACTION ANALYSIS

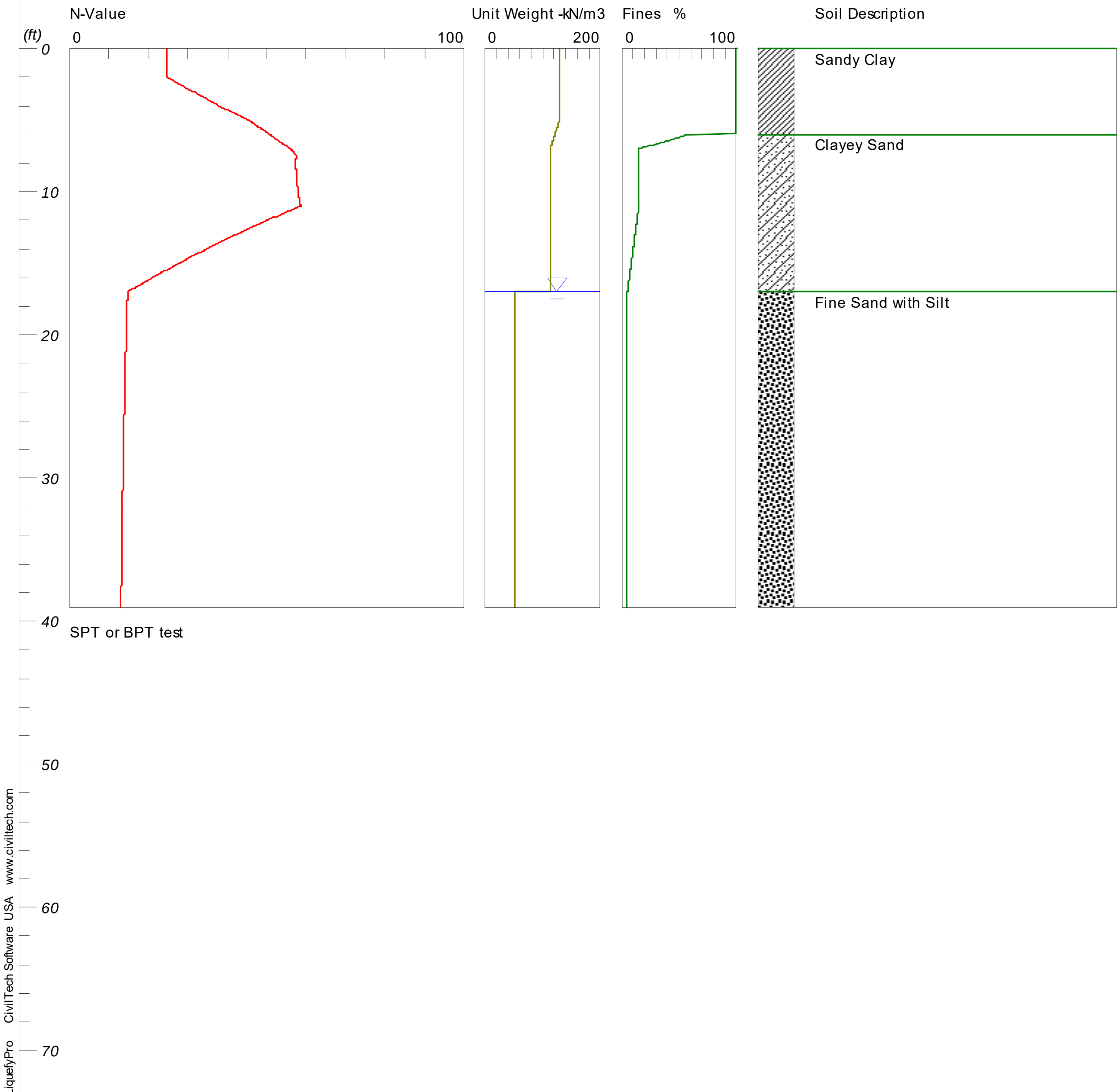
SM10391.2 VALLEMAR ST MOSS BEACH

Hole No.=3 Water Depth=17 ft Surface Elev.=56.5

Ground Improvement of Fill=7.5 ft

Magnitude=8

Acceleration=.896g

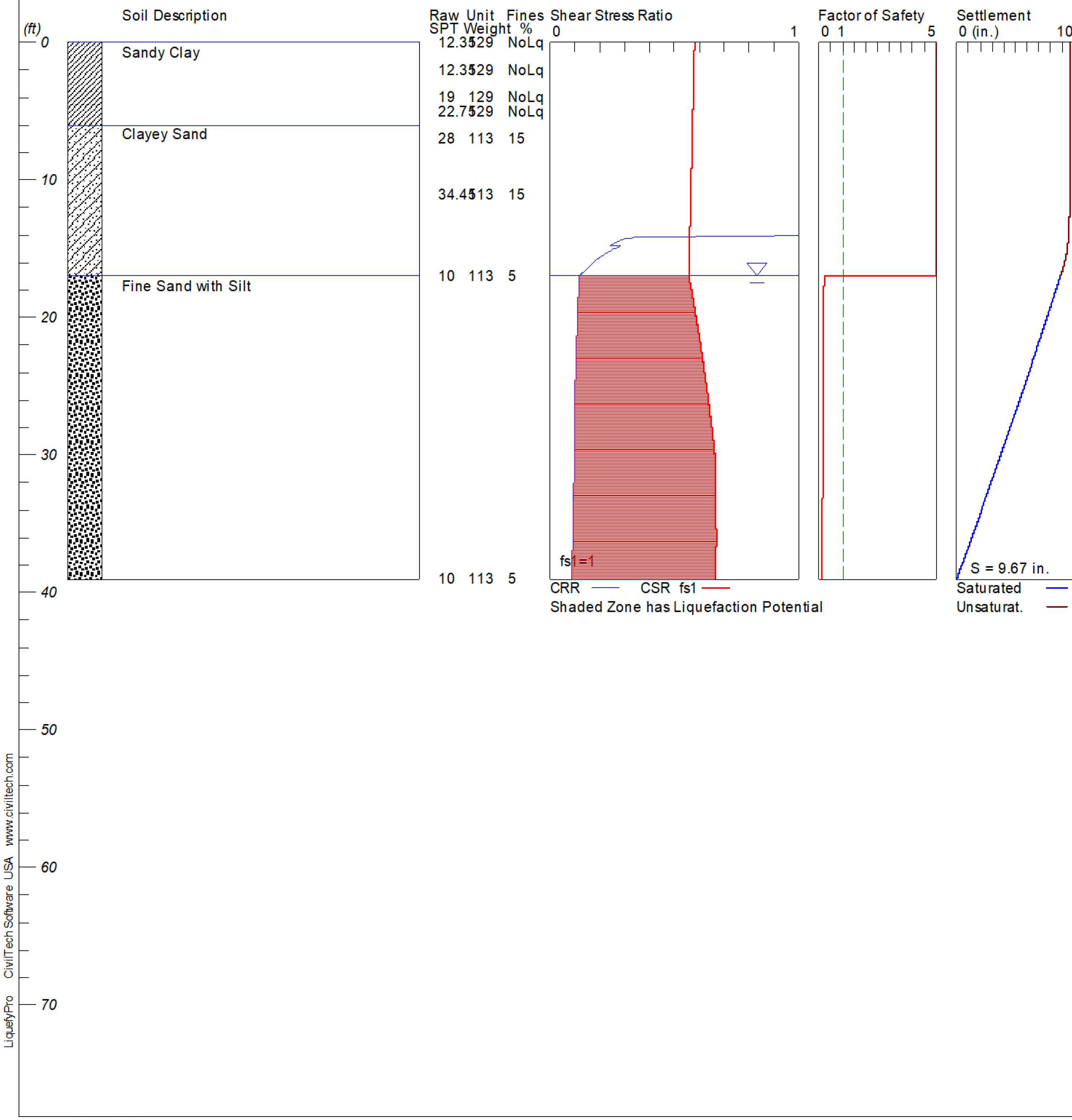


LIQUEFACTION ANALYSIS

SM10391.2 VALLEMAR ST MOSS BEACH

Hole No.=3 Water Depth=17 ft Surface Elev.=56.5 ***Magnitude=8***

Ground Improvement of Fill=7.5 ft ***Acceleration=.896g***



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                                    LIQUEFACTION ANALYSIS CALCULATION DETAILS

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        Licensed to ,   6/13/2018       4:42:23 PM

        Input File Name: C:\Users\Moses\Documents\Projects\San Mateo\Coastal
Bluff\10391 Vallemor Bluff\2018 Update\Liquefaction\liquefaction B-3.liq
        Title:  SM10391.2 VALLEMAR ST MOSS BEACH
        Subtitle:  BORING B-3

 Input Data:
        Surface Elev.=56.5
        Hole No.=3
        Depth of Hole=39.00 ft
        Water Table during Earthquake= 17.00 ft
        Water Table during In-Situ Testing= 17.00 ft
        Max. Acceleration=0.9 g
        Earthquake Magnitude=8.00
        No-Liquefiable Soils:   CL, OL are Non-Liq. Soil
        1. SPT or BPT Calculation.
        2. Settlement Analysis Method: Ishihara / Yoshimine
        3. Fines Correction for Liquefaction: Modify Stark/Olson
        4. Fine Correction for Settlement: During Liquefaction*
        5. Settlement Calculation in: All zones*
        6. Hammer Energy Ratio,                                  Ce = .89
        7. Borehole Diameter,                                        Cb= 1
        8. Sampling Method,                                         Cs= 1
        9. User request factor of safety (apply to CSR) ,    User= 1
           Plot one CSR curve (fs1=1)
        10. Average two input data between two Depths: Yes*
        * Recommended Options

        Fill on Ground Surface= 7.5 ft          Fill Unit Weight= 125 pcf
        Factor of soil strength (SPT or CPT) change due to fill= 1
        Depth of this report is based on original ground surface, not based on fill
          1 atm (atmosphere) = 1 tsf (ton/ft2)

        In-Situ Test Data:
        Depth   SPT     Gamma   Fines
```

ft		pcf	%
0.00	12.35	129.00	NoLiq
2.00	12.35	129.00	NoLiq
4.00	19.00	129.00	NoLiq
5.00	22.75	129.00	NoLiq
7.00	28.00	113.00	15.00
11.00	34.45	113.00	15.00
17.00	10.00	113.00	5.00
39.00	10.00	113.00	5.00

Output Results:
Calculation segment, dz=0.050 ft
User defined Print Interval, dp=5.00 ft

Peak Ground Acceleration (PGA), a_max = 0.90g

CSR Calculation:

Depth	gamma	sigma	gamma'	sigma'	rd	mZ	a(z)	CSR	x fs1	=CSRfs
ft	pcf	atm	pcf	atm		g	g			
0.00	129.00	0.443	129.00	0.443	1.00	0.000	0.896	0.58	1.00	0.58
5.00	129.00	0.748	129.00	0.748	0.99	0.000	0.896	0.58	1.00	0.58
10.00	113.00	1.023	113.00	1.023	0.98	0.000	0.896	0.57	1.00	0.57
15.00	113.00	1.290	113.00	1.290	0.97	0.000	0.896	0.56	1.00	0.56
20.00	113.00	1.557	50.60	1.468	0.95	0.000	0.896	0.59	1.00	0.59
25.00	113.00	1.824	50.60	1.588	0.94	0.000	0.896	0.63	1.00	0.63
30.00	113.00	2.091	50.60	1.707	0.93	0.000	0.896	0.66	1.00	0.66
35.00	113.00	2.358	50.60	1.827	0.89	0.000	0.896	0.67	1.00	0.67

CSR is based on water table at 17.00 during earthquake
sigma and sigma' are based on fill on ground surface during earthquake

CRR Calculation from SPT or BPT data:

Depth ft	SPT	Cebs	Cr	sigma' atm	Cn	(N1)60	Fines %	d(N1)60	(N1)60f	CRR7.5
0.00	24.70	0.89	0.75	0.000	1.70	28.03	NoLiq	22.80	50.83	2.00
5.00	45.50	0.89	0.75	0.305	1.70	51.63	NoLiq	22.80	74.43	2.00
10.00	57.94	0.89	0.85	0.580	1.31	57.58	15.00	2.40	59.98	2.00
15.00	27.65	0.89	0.95	0.847	1.09	25.41	8.33	0.80	26.21	0.30
20.00	14.32	0.89	0.95	1.025	0.99	11.96	5.00	0.00	11.96	0.13
25.00	13.87	0.89	0.95	1.145	0.93	10.96	5.00	0.00	10.96	0.12
30.00	13.50	0.89	1.00	1.264	0.89	10.69	5.00	0.00	10.69	0.12
35.00	13.20	0.89	1.00	1.384	0.85	9.99	5.00	0.00	9.99	0.11

CRR is based on water table at 17.00 during In-Situ Testing
SPT or CPT are increased due to increased overburden pressure

Factor of Safety, - Earthquake Magnitude= 8.00:

Depth ft	sigC' atm	CRR7.5	x Ksig	=CRRv	x MSF	=CRRm	CSRfs	F.S.=CRRm/CSRfs
0.00	0.00	2.00	1.00	2.00	0.85	2.00	0.58	5.00 ^
5.00	0.20	2.00	1.00	2.00	0.85	2.00	0.58	5.00 ^
10.00	0.38	2.00	1.00	2.00	0.85	1.69	0.57	5.00
15.00	0.55	0.30	1.00	0.30	0.85	0.26	0.56	5.00
20.00	0.67	0.13	1.00	0.13	0.85	0.11	0.59	0.19 *
25.00	0.74	0.12	1.00	0.12	0.85	0.10	0.63	0.16 *
30.00	0.82	0.12	1.00	0.12	0.85	0.10	0.66	0.15 *
35.00	0.90	0.11	1.00	0.11	0.85	0.09	0.67	0.14 *

* F.S.<1: Liquefaction Potential Zone. (If above water table: F.S.=5)
^ No-liquefiable Soils or above Water Table.
(F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)

CPT convert to SPT for Settlement Analysis:
Fines Correction for Settlement Analysis:

Depth ft	Ic	qc/N60	qc1 atm	(N1)60	Fines %	d(N1)60	(N1)60s
0.00	-	-	-	50.83	NoLiq	0.00	50.83
5.00	-	-	-	74.43	NoLiq	0.00	74.43
10.00	-	-	-	59.98	15.00	0.00	59.98
15.00	-	-	-	26.21	8.33	0.00	26.21
20.00	-	-	-	11.96	5.00	0.00	11.96
25.00	-	-	-	10.96	5.00	0.00	10.96
30.00	-	-	-	10.69	5.00	0.00	10.69
35.00	-	-	-	9.99	5.00	0.00	9.99

(N1)60s has been fines corrected in liquefaction analysis, therefore d(N1)60=0.
Fines=NoLiq means the soils are not liquefiable.

Settlement of Saturated Sands:
Settlement Analysis Method: Ishihara / Yoshimine

Depth ft	CSRsf	/ MSF*	=CSRm	F.S.	Fines %	(N1)60s	Dr %	ec %	dsz in.	dsp in.	S in.
38.95	0.67	1.00	0.67	0.13	5.00	9.51	49.36	3.583	2.1E-2	0.021	0.021
35.00	0.67	1.00	0.67	0.14	5.00	9.99	50.51	3.482	2.1E-2	1.673	1.694
30.00	0.66	1.00	0.66	0.15	5.00	10.69	52.16	3.364	2.0E-2	2.054	3.748
25.00	0.63	1.00	0.63	0.16	5.00	10.96	52.79	3.319	2.0E-2	2.010	5.758
20.00	0.59	1.00	0.59	0.19	5.00	11.96	55.02	3.160	1.9E-2	1.945	7.703
17.00	0.56	1.00	0.56	0.21	5.00	12.68	56.58	3.049	1.8E-2	1.118	8.821

Settlement of Saturated Sands=8.821 in.
qc1 and (N1)60 is after fines correction in liquefaction analysis
dsz is per each segment, dz=0.05 ft
dsp is per each print interval, dp=5.00 ft
S is cumulated settlement at this depth

Settlement of Unsaturated Sands:

Depth	sigma'	sigC'	(N1)60s	CSRsf	Gmax	g*Ge/Gm	g_eff	ec7.5	Cec	ec	dsz	dsp	S
ft	atm	atm			atm			%		%	in.	in.	in.
16.95	0.95	0.62	12.99	0.56	825.72	6.4E-4	1.0000	1.7172	1.15	1.9811	2.38E-2	0.024	0.024
15.00	0.85	0.55	26.21	0.56	984.30	4.8E-4	0.8291	0.5723	1.15	0.6603	7.92E-3	0.575	0.599
10.00	0.58	0.38	59.98	0.57	1072.95	3.1E-4	0.0950	0.0300	1.15	0.0346	4.16E-4	0.208	0.808
5.00	0.30	0.20	74.43	0.58	836.13	2.1E-4	0.0419	0.0133	1.15	0.0153	0.00E0	0.045	0.853
0.00	0.00	0.00	50.83	0.58	4.22	1.4E-6	0.0010	0.0003	1.15	0.0004	0.00E0	0.000	0.853

Settlement of Unsaturated Sands=0.853 in.
dsz is per each segment, dz=0.05 ft
dsp is per each print interval, dp=5.00 ft
S is cumulated settlement at this depth

Total Settlement of Saturated and Unsaturated Sands=9.674 in.
Differential Settlement=4.837 to 6.385 in.

Units: Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight = pcf; Depth = ft; Settlement = in.

1 atm (atmosphere) = 1.0581 tsf(1 tsf = 1 ton/ft2 = 2 kip/ft2)
1 atm (atmosphere) = 101.325 kPa(1 kPa = 1 kN/m2 = 0.001 Mpa)

SPT	Field data from Standard Penetration Test (SPT)
BPT	Field data from Becker Penetration Test (BPT)
qc	Field data from Cone Penetration Test (CPT) [atm (tsf)]
fs	Friction from CPT testing [atm (tsf)]
Rf	Ratio of fs/qc (%)
gamma	Total unit weight of soil
gamma'	Effective unit weight of soil
Fines	Fines content [%]
D50	Mean grain size
Dr	Relative Density
sigma	Total vertical stress [atm]

```
        sigma'          Effective vertical stress [atm]
        sigC'           Effective confining pressure [atm]
        rd              Acceleration reduction coefficient by Seed
        a_max.          Peak Ground Acceleration (PGA) in ground surface
        mZ              Linear acceleration reduction coefficient X depth
        a_min.          Minimum acceleration under linear reduction, mZ
        CRRv            CRR after overburden stress correction, CRRv=CRR7.5 * Ksig
          CRR7.5                Cyclic resistance ratio (M=7.5)
          Ksig          Overburden stress correction factor for CRR7.5
        CRRm            After magnitude scaling correction CRRm=CRRv * MSF
          MSF           Magnitude scaling factor from M=7.5 to user input M
        CSR             Cyclic stress ratio induced by earthquake
        CSRfs           CSRfs=CSR*fs1 (Default fs1=1)
          fs1           First CSR curve in graphic defined in #9 of Advanced page
          fs2           2nd CSR curve in graphic defined in #9 of Advanced page
        F.S.            Calculated factor of safety against liquefaction
F.S.=CRRm/CSRsf
        Cebs            Energy Ratio, Borehole Dia., and Sampling Method
Corrections
        Cr              Rod Length Corrections
        Cn              Overburden Pressure Correction
        (N1)60          SPT after corrections, (N1)60=SPT * Cr * Cn * Cebs
        d(N1)60         Fines correction of SPT
        (N1)60f         (N1)60 after fines corrections, (N1)60f=(N1)60 + d(N1)60
        Cq              Overburden stress correction factor
        qc1             CPT after Overburden stress correction
        dqc1            Fines correction of CPT
        qc1f            CPT after Fines and Overburden correction, qc1f=qc1 + dqc1
        qc1n            CPT after normalization in Robertson's method
        Kc              Fine correction factor in Robertson's Method
        qc1f            CPT after Fines correction in Robertson's Method
        Ic              Soil type index in Suzuki's and Robertson's Methods
        (N1)60s         (N1)60 after settlement fines corrections
        CSRm            After magnitude scaling correction for Settlement
calculation  CSRm=CSRsf / MSF*
          CSRfs                 Cyclic stress ratio induced by earthquake with user
inputed fs
          MSF*                  Scaling factor from CSR, MSF*=1, based on Item 2 of
Page C.
        ec              Volumetric strain for saturated sands
        dz              Calculation segment, dz=0.050 ft
        dsz             Settlement in each segment, dz
        dp              User defined print interval
        dsp             Settlement in each print interval, dp
        Gmax            Shear Modulus at low strain
        g_eff           gamma_eff, Effective shear Strain
        g*Ge/Gm         gamma_eff * G_eff/G_max,        Strain-modulus ratio
        ec7.5           Volumetric Strain for magnitude=7.5
```

Cec Magnitude correction factor for any magnitude
ec Volumetric strain for unsaturated sands, ec=Cec * ec7.5
NoLiq No-Liquefy Soils

References:

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1. NCEER Workshop on Evaluation of Liquefaction Resistance of Soils. Youd, T.L., and Idriss, I.M., eds., Technical Report NCEER 97-0022.
SP117. Southern California Earthquake Center. Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California. University of Southern California. March 1999.
2. RECENT ADVANCES IN SOIL LIQUEFACTION ENGINEERING AND SEISMIC SITE RESPONSE EVALUATION, Paper No. SPL-2, PROCEEDINGS: Fourth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, San Diego, CA, March 2001.
3. RECENT ADVANCES IN SOIL LIQUEFACTION ENGINEERING: A UNIFIED AND CONSISTENT FRAMEWORK, Earthquake Engineering Research Center, Report No. EERC 2003-06 by R.B Seed and etc. April 2003.

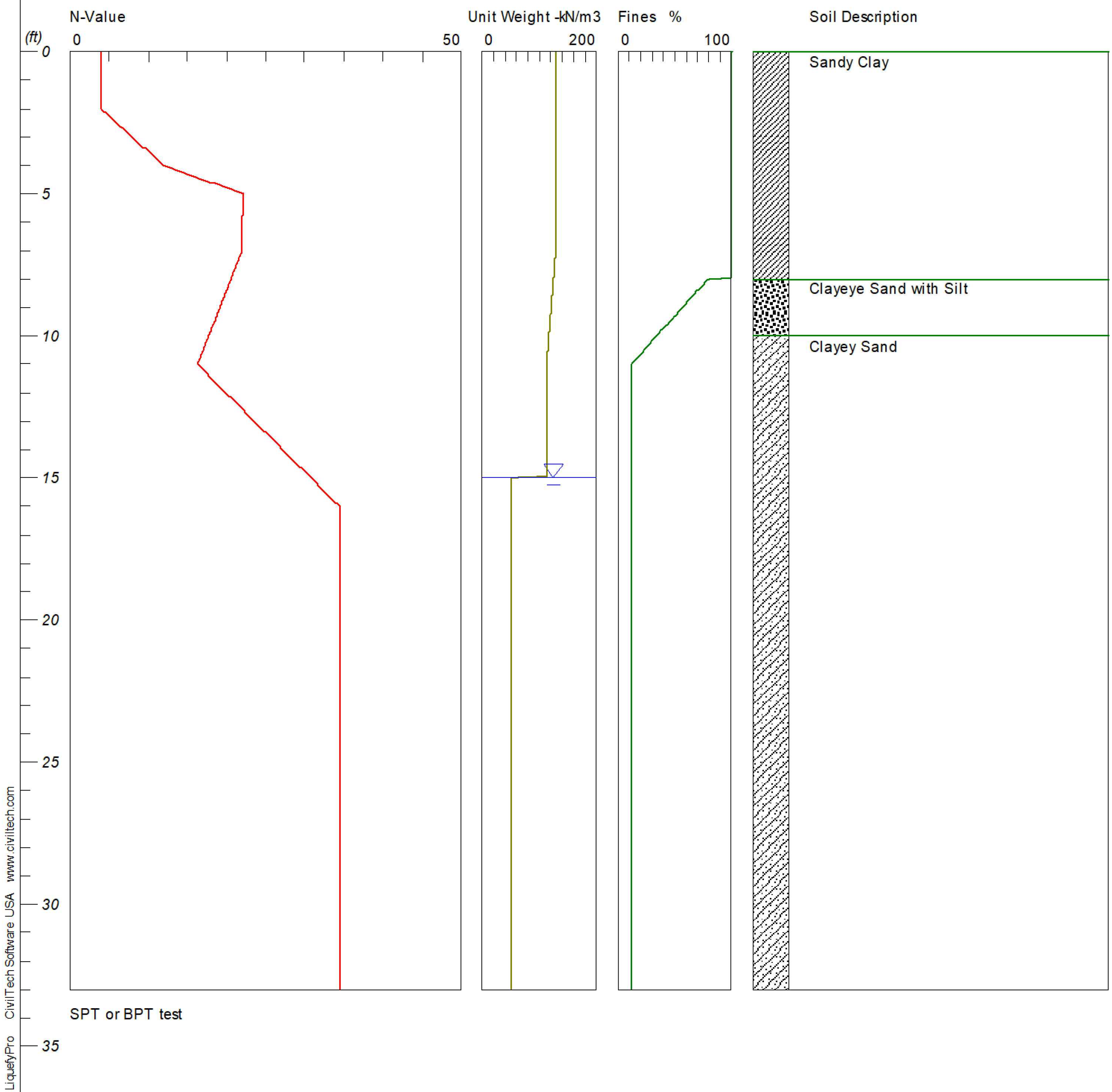
Note: Print Interval you selected does not show complete results. To get complete results, you should select 'Segment' in Print Interval (Item 12, Page C).

LIQUEFACTION ANALYSIS

SM10391.2 VALLEMAR ST MOSS BEACH

Hole No.=4   Water Depth=15 ft   Surface Elev.=50.3

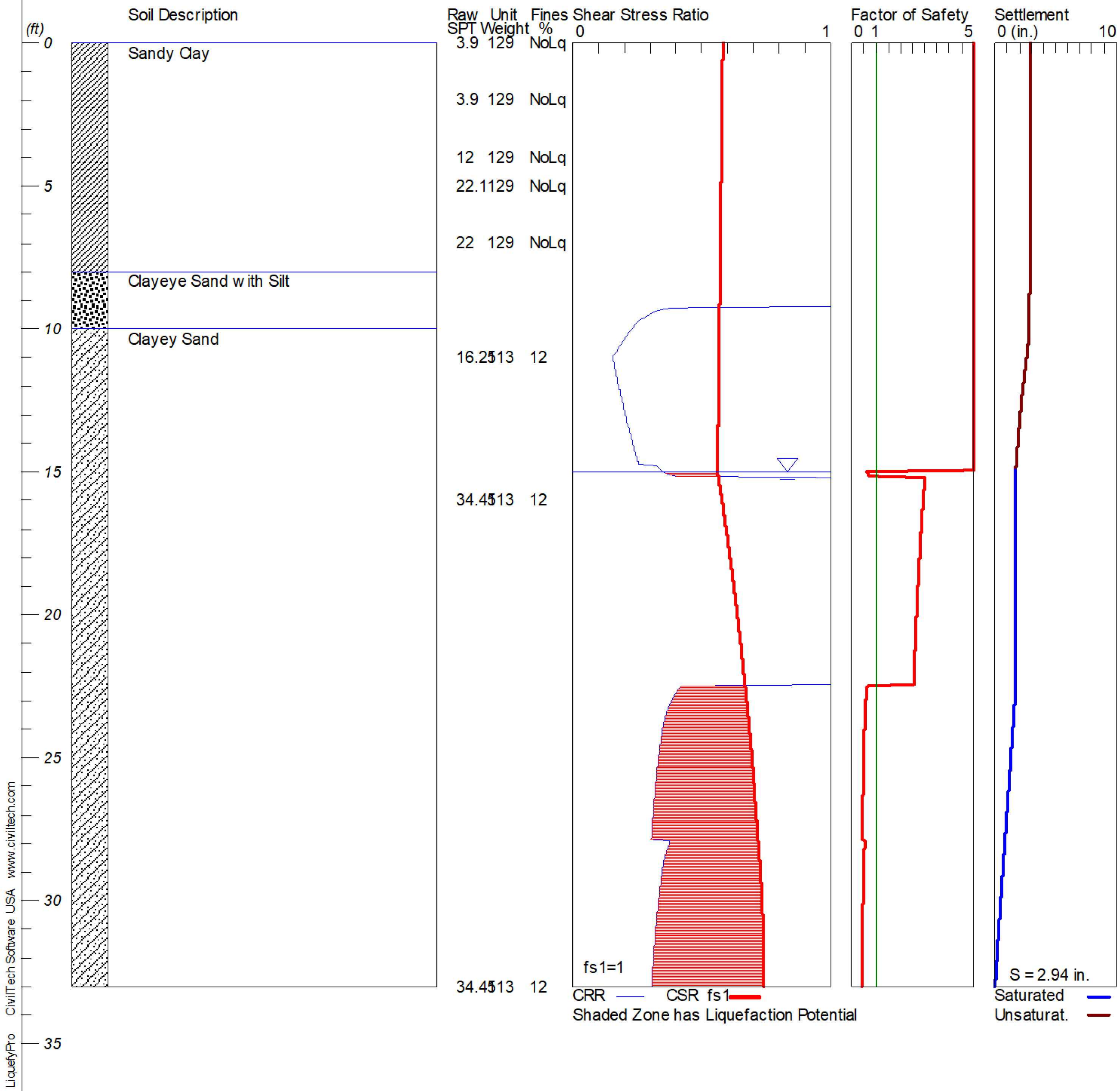
Magnitude=8
Acceleration=.896g



LIQUEFACTION ANALYSIS
SM10391.2 VALLEMAR ST MOSS BEACH

Hole No.=4   Water Depth=15 ft   Surface Elev.=50.3

Magnitude=8
Acceleration=.896g



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LIQUEFACTION ANALYSIS CALCULATION DETAILS

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Input File Name: C:\Users\Moses\Documents\Projects\San Mateo\Coastal Bluff\10391 Vallemor Bluff\2018 Update\Liquefaction\liquefaction B-4.liq
Title: SM10391.2 VALLEMAR ST MOSS BEACH
Subtitle: BORING B-4

Input Data:
Surface Elev.=50.3
Hole No.=4
Depth of Hole=33.00 ft
Water Table during Earthquake= 15.00 ft
Water Table during In-Situ Testing= 15.00 ft
Max. Acceleration=0.9 g
Earthquake Magnitude=8.00
No-Liquefiable Soils: CL, OL are Non-Liq. Soil
1. SPT or BPT Calculation.
2. Settlement Analysis Method: Ishihara / Yoshimine
3. Fines Correction for Liquefaction: Modify Stark/Olson
4. Fine Correction for Settlement: During Liquefaction*
5. Settlement Calculation in: All zones*
6. Hammer Energy Ratio,   Ce = .89
7. Borehole Diameter,   Cb= 1
8. Sampling Method,   Cs= 1
9. User request factor of safety (apply to CSR) ,   User= 1
   Plot one CSR curve (fs1=1)
10. Average two input data between two Depths: Yes*
* Recommended Options

In-Situ Test Data:

Depth ft	SPT	Gamma pcf	Fines %
0.00	3.90	129.00	NoLiq
2.00	3.90	129.00	NoLiq
4.00	12.00	129.00	NoLiq

5.00	22.10	129.00	NoLiq
7.00	22.00	129.00	NoLiq
11.00	16.25	113.00	12.00
16.00	34.45	113.00	12.00
33.00	34.45	113.00	12.00

Output Results:
Calculation segment, dz=0.050 ft
User defined Print Interval, dp=5.00 ft

Peak Ground Acceleration (PGA), a_max = 0.90g

CSR Calculation:

Depth ft	gamma pcf	sigma atm	gamma' pcf	sigma' atm	rd	mZ g	a(z) g	CSR	x fs1	=CSRfs
0.00	129.00	0.000	129.00	0.000	1.00	0.000	0.896	0.58	1.00	0.58
5.00	129.00	0.305	129.00	0.305	0.99	0.000	0.896	0.58	1.00	0.58
10.00	117.00	0.601	117.00	0.601	0.98	0.000	0.896	0.57	1.00	0.57
15.00	113.00	0.869	50.60	0.869	0.97	0.000	0.896	0.56	1.00	0.56
20.00	113.00	1.136	50.60	0.989	0.95	0.000	0.896	0.64	1.00	0.64
25.00	113.00	1.403	50.60	1.108	0.94	0.000	0.896	0.69	1.00	0.69
30.00	113.00	1.670	50.60	1.228	0.93	0.000	0.896	0.74	1.00	0.74

CSR is based on water table at 15.00 during earthquake

CRR Calculation from SPT or BPT data:

Depth ft	SPT	Cebs	Cr	sigma' atm	Cn	(N1)60	Fines %	d(N1)60	(N1)60f	CRR7.5
0.00	3.90	0.89	0.75	0.000	1.70	4.43	NoLiq	22.80		

27.23	0.32							
5.00	22.10	0.89	0.75	0.305	1.70	25.08	NoLiq	22.80
47.88	2.00							
10.00	17.69	0.89	0.85	0.601	1.29	17.26	34.25	7.02
24.28	0.27							
15.00	30.81	0.89	0.95	0.869	1.07	27.94	12.00	1.68
29.62	0.41							
20.00	34.45	0.89	0.95	0.989	1.01	29.29	12.00	1.68
30.97	2.00							
25.00	34.45	0.89	0.95	1.108	0.95	27.67	12.00	1.68
29.35	0.39							
30.00	34.45	0.89	1.00	1.228	0.90	27.67	12.00	1.68
29.35	0.39							

CRR is based on water table at 15.00 during In-Situ Testing

Factor of Safety, - Earthquake Magnitude= 8.00:

Depth ft	sigC' atm	CRR7.5	x Ksig	=CRRv	x MSF	=CRRm	CSRfs	F.S.=CRRm/CSRfs
0.00	0.00	0.32	1.00	0.32	0.85	2.00	0.58	5.00 ^
5.00	0.20	2.00	1.00	2.00	0.85	2.00	0.58	5.00 ^
10.00	0.39	0.27	1.00	0.27	0.85	0.23	0.57	5.00
15.00	0.56	0.41	1.00	0.41	0.85	0.35	0.56	0.62 *
20.00	0.64	2.00	1.00	2.00	0.85	1.69	0.64	2.66
25.00	0.72	0.39	1.00	0.39	0.85	0.33	0.69	0.48 *
30.00	0.80	0.39	1.00	0.39	0.85	0.33	0.74	0.45 *

* F.S.<1: Liquefaction Potential Zone. (If above water table: F.S.=5)
^ No-liquefiable Soils or above Water Table.
(F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)

CPT convert to SPT for Settlement Analysis:
Fines Correction for Settlement Analysis:

Depth ft	Ic	qc/N60	qc1 atm	(N1)60	Fines %	d(N1)60	(N1)60s
0.00	-	-	-	27.23	NoLiq	0.00	27.23
5.00	-	-	-	47.88	NoLiq	0.00	47.88
10.00	-	-	-	24.28	34.25	0.00	24.28
15.00	-	-	-	29.62	12.00	0.00	29.62
20.00	-	-	-	30.97	12.00	0.00	30.97
25.00	-	-	-	29.35	12.00	0.00	29.35
30.00	-	-	-	29.35	12.00	0.00	29.35

(N1)60s has been fines corrected in liquefaction analysis, therefore d(N1)60=0.
Fines=NoLiq means the soils are not liquefiable.

Settlement of Saturated Sands:
Settlement Analysis Method: Ishihara / Yoshimine

Depth	CSRsf	/ MSF*	=CSRm	F.S.	Fines	(N1)60s	Dr	ec	dsz	dsp	S
ft					%		%	%	in.	in.	in.
32.95	0.74	1.00	0.74	0.41	12.00	28.59	86.95	1.450	8.7E-3	0.009	0.009
30.00	0.74	1.00	0.74	0.45	12.00	29.35	88.61	1.370	8.2E-3	0.499	0.508
25.00	0.69	1.00	0.69	0.48	12.00	29.35	88.60	1.364	8.2E-3	0.826	1.334
20.00	0.64	1.00	0.64	2.66	12.00	30.97	92.30	0.000	0.0E0	0.376	1.710
15.00	0.56	1.00	0.56	0.62	12.00	29.62	89.21	1.127	6.8E-3	0.025	1.735

Settlement of Saturated Sands=1.735 in.
qc1 and (N1)60 is after fines correction in liquefaction analysis
dsz is per each segment, dz=0.05 ft
dsp is per each print interval, dp=5.00 ft
S is cumulated settlement at this depth

Settlement of Unsaturated Sands:

Depth	sigma'	sigC'	(N1)60s	CSRsf	Gmax	g*Ge/Gm	g_eff	ec7.5	Cec	ec	dsz	dsp	S
ft	atm	atm			atm			%		%	in.	in.	in.
14.95	0.87	0.56	29.50	0.56	1035.88	4.7E-4	0.6955	0.4060	1.15	0.4684	5.62E-3	0.006	0.006
10.00	0.60	0.39	24.28	0.57	808.65	4.2E-4	0.3758	0.2873	1.15	0.3315	3.98E-3	1.051	1.056
5.00	0.30	0.20	47.88	0.58	721.88	2.4E-4	0.0655	0.0207	1.15	0.0239	0.00E0	0.153	1.209

```
        0.00    0.00    0.00    27.23   0.58    3.43    1.7E-6  0.0010  0.0007
1.15    0.0008  0.00E0  0.000   1.209
```

        Settlement of Unsaturated Sands=1.209 in.
        dsz is per each segment, dz=0.05 ft
        dsp is per each print interval, dp=5.00 ft
        S is cumulated settlement at this depth

        Total Settlement of Saturated and Unsaturated Sands=2.944 in.
        Differential Settlement=1.472 to 1.943 in.

        Units: Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight = pcf; Depth = ft; Settlement = in.

```
        1 atm (atmosphere) = 1.0581 tsf(1 tsf = 1 ton/ft2 = 2 kip/ft2)
        1 atm (atmosphere) = 101.325 kPa(1 kPa = 1 kN/m2 = 0.001 Mpa)
        SPT             Field data from Standard Penetration Test (SPT)
        BPT             Field data from Becker Penetration Test (BPT)
        qc              Field data from Cone Penetration Test (CPT) [atm (tsf)]
        fs              Friction from CPT testing [atm (tsf)]
        Rf              Ratio of fs/qc (%)
        gamma           Total unit weight of soil
        gamma'          Effective unit weight of soil
        Fines           Fines content [%]
        D50             Mean grain size
        Dr              Relative Density
        sigma           Total vertical stress [atm]
        sigma'          Effective vertical stress [atm]
        sigC'           Effective confining pressure [atm]
        rd              Acceleration reduction coefficient by Seed
        a_max.          Peak Ground Acceleration (PGA) in ground surface
        mZ              Linear acceleration reduction coefficient X depth
        a_min.          Minimum acceleration under linear reduction, mZ
        CRRv            CRR after overburden stress correction, CRRv=CRR7.5 * Ksig
          CRR7.5                Cyclic resistance ratio (M=7.5)
          Ksig          Overburden stress correction factor for CRR7.5
        CRRm            After magnitude scaling correction CRRm=CRRv * MSF
          MSF           Magnitude scaling factor from M=7.5 to user input M
        CSR             Cyclic stress ratio induced by earthquake
        CSRfs           CSRfs=CSR*fs1 (Default fs1=1)
          fs1           First CSR curve in graphic defined in #9 of Advanced page
          fs2           2nd CSR curve in graphic defined in #9 of Advanced page
        F.S.            Calculated factor of safety against liquefaction
F.S.=CRRm/CSRsf
```

Cebs Energy Ratio, Borehole Dia., and Sampling Method Corrections
Cr Rod Length Corrections
Cn Overburden Pressure Correction
(N1)60 SPT after corrections, (N1)60=SPT * Cr * Cn * Cebs
d(N1)60 Fines correction of SPT
(N1)60f (N1)60 after fines corrections, (N1)60f=(N1)60 + d(N1)60
Cq Overburden stress correction factor
qc1 CPT after Overburden stress correction
dqc1 Fines correction of CPT
qc1f CPT after Fines and Overburden correction, qc1f=qc1 + dqc1
qc1n CPT after normalization in Robertson's method
Kc Fine correction factor in Robertson's Method
qc1f CPT after Fines correction in Robertson's Method
Ic Soil type index in Suzuki's and Robertson's Methods
(N1)60s (N1)60 after settlement fines corrections
CSRm After magnitude scaling correction for Settlement calculation CSRm=CSRsf / MSF*
CSRfs Cyclic stress ratio induced by earthquake with user inputed fs
MSF* Scaling factor from CSR, MSF*=1, based on Item 2 of Page C.
ec Volumetric strain for saturated sands
dz Calculation segment, dz=0.050 ft
dsz Settlement in each segment, dz
dp User defined print interval
dsp Settlement in each print interval, dp
Gmax Shear Modulus at low strain
g_eff gamma_eff, Effective shear Strain
g*Ge/Gm gamma_eff * G_eff/G_max, Strain-modulus ratio
ec7.5 Volumetric Strain for magnitude=7.5
Cec Magnitude correction factor for any magnitude
ec Volumetric strain for unsaturated sands, ec=Cec * ec7.5
NoLiq No-Liquefy Soils

References:

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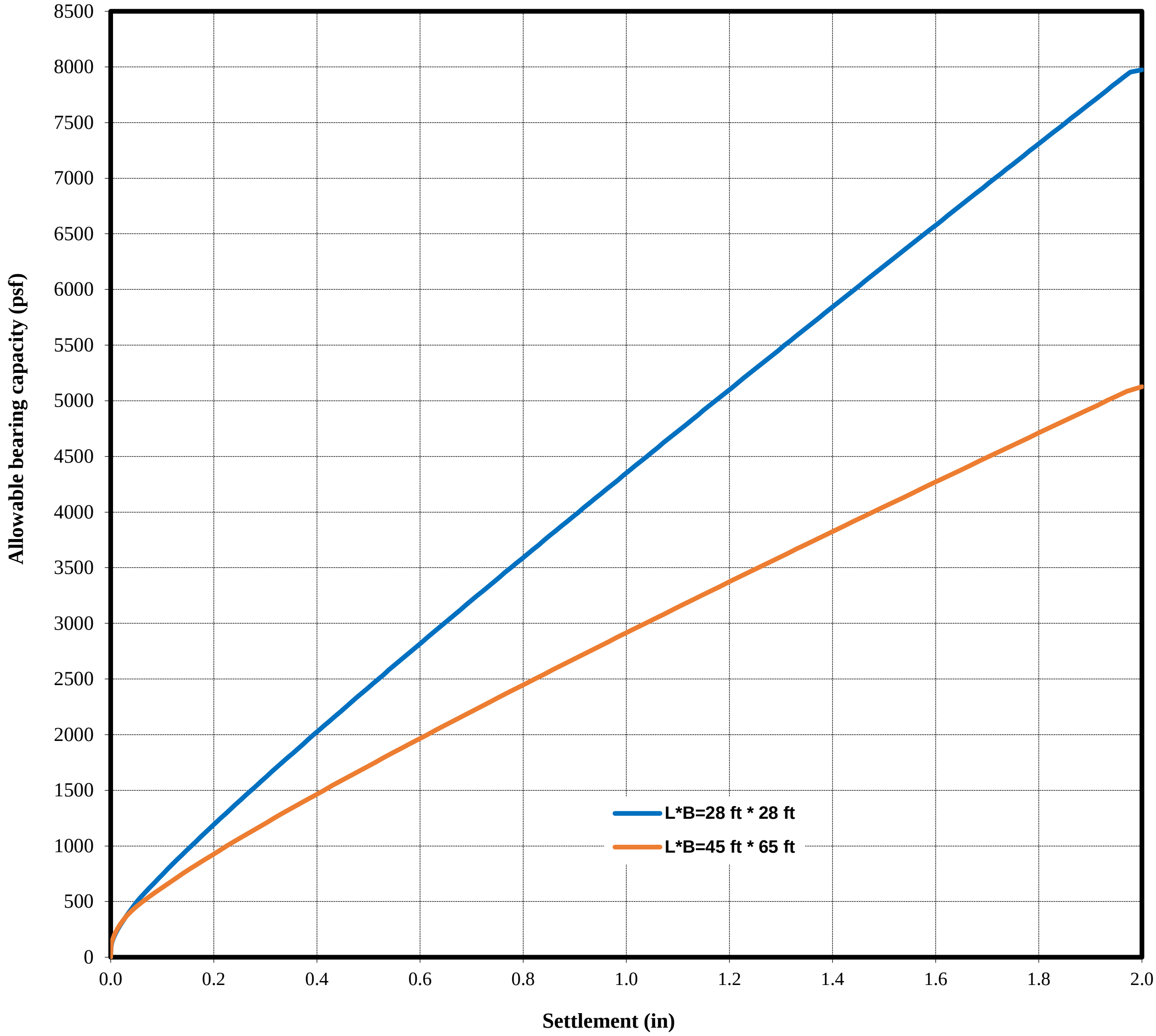
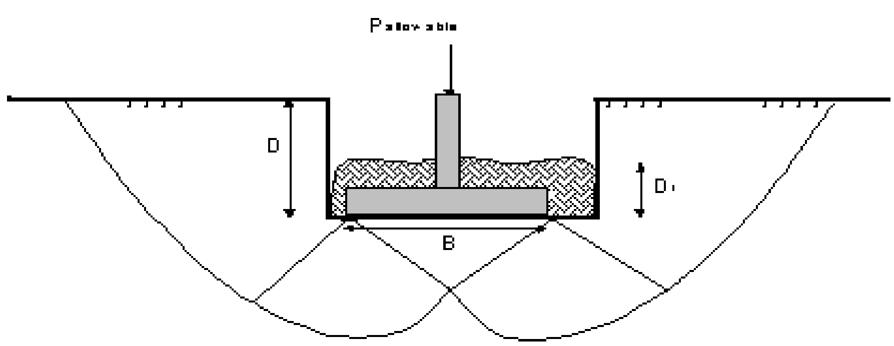
1. NCEER Workshop on Evaluation of Liquefaction Resistance of Soils. Youd, T.L., and Idriss, I.M., eds., Technical Report NCEER 97-0022.
SP117. Southern California Earthquake Center. Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California. University of Southern California. March 1999.
2. RECENT ADVANCES IN SOIL LIQUEFACTION ENGINEERING AND SEISMIC SITE RESPONSE EVALUATION, Paper No. SPL-2, PROCEEDINGS: Fourth International Conference on Recent Advances in Geotechnical Earthquake

Engineering and Soil Dynamics, San Diego, CA, March 2001.
3. RECENT ADVANCES IN SOIL LIQUEFACTION ENGINEERING: A UNIFIED AND CONSISTENT FRAMEWORK, Earthquake Engineering Research Center, Report No. EERC 2003-06 by R.B Seed and etc. April 2003.

Note: Print Interval you selected does not show complete results. To get complete results, you should select 'Segment' in Print Interval (Item 12, Page C).

Allowable bearing capacity for Mat Foundation



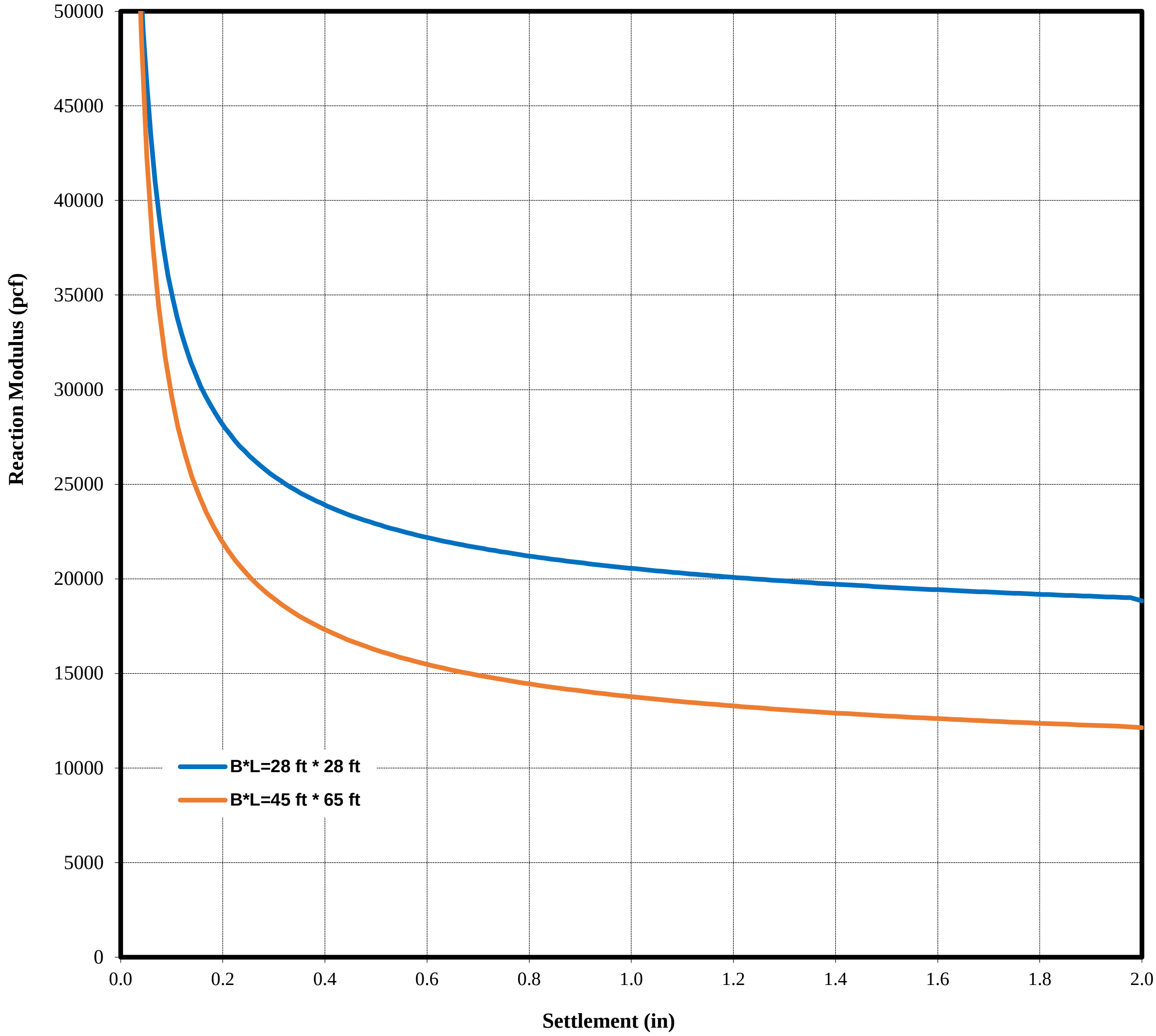


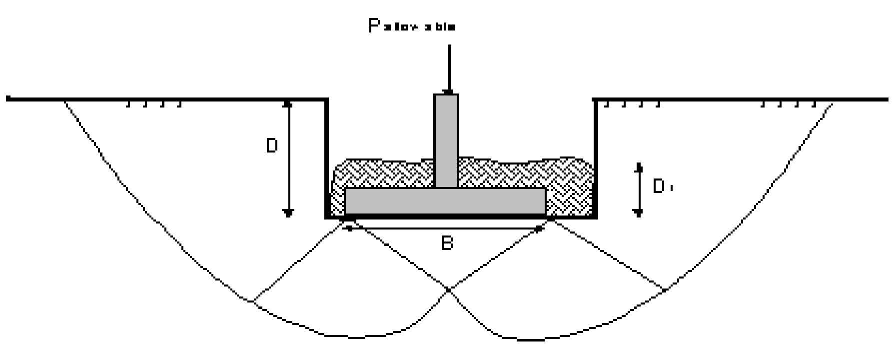
D= 1.0 ft
Df= 1.0 ft

Notes:
D : Depth of footing with respect to ground surface
Df :Depth of footing embedment

Allowable settlement = 2 inches

Soil Reaction Modulus for Mat Foundation





D= 1.0 ft
Df= 1.0 ft

Notes:
D : Depth of footing with respect to ground surface
Df :Depth of footing embedment

Allowable settlement = 2 inches