GEOTECHNICAL INVESTIGATION
For
Five Home Residential Development
Vallemar Street and Juliana Avenue
APN 037-086-23, -26, -27, -28, -29
Lots A, D, E, F, G
Moss Beach, California

Prepared For
Moss Beach Associates, LLC
c/o Lawlor LandUse Manager, Santa Cruz, California

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

Project No. SM10391.2
August 2016

Project No. SM10391.2
12 August 2016

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

Subject: Geotechnical Investigation

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:

In accordance with your authorization, we have performed a Geotechnical Investigation for the referenced property in Moss Beach, California. This investigation was completed with consideration to the Coastal Bluff Recession Map prepared by our firm for the referenced property. The Coastal Bluff Recession Map is included in the appendix of this report and should be reviewed as part of this document.

The accompanying report presents our geotechnical recommendations and design criteria, along with the results and methodology of our investigation. If the recommendations in our geotechnical report are followed during project design and construction, the project will be subject to "ordinary risks" as defined in the Scale of Acceptable Risks From Geologic Hazards" in Appendix E of this report. If this level of risk is unacceptable, more extensive mitigation of the hazards can be recommended. In brief we have recommended the new residences be situated landward of the estimated 50 year coastal bluff recession setback and supported by conventional spread foundations embedded into an earthen mat of engineered fill.

If you have any questions concerning our conclusions or recommendations, presented in this report please contact our office.

Respectfully Submitted,

HARO, KASUNICH AND ASSOCIATES, INC.

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TABLE OF CONTENTS

Table of Contents Cont'd

Project No. SM10391.2
12 August 2016

GEOTECHNICAL INVESTIGATION

Introduction

This report presents the results of our Geotechnical Investigation for the proposed construction of five (5) new single family dwellings on five separate parcels. The five parcels are designated on preliminary maps as lots A, D, E, F, & G. Lots B and C are designated as open space. We have also prepared an estimated 50 year future coastal bluff recession map for the bluff that borders Lots A, B, C, & D. The recession map should be reviewed in conjunction with this geotechnical investigation.

Purpose and Scope

The purpose of the geotechnical investigation is to develop geotechnical design parameters for design and construction of the new residences at the referenced site. We performed slope stability analysis on a critical cross section selected by HKA and developed from a topographic map prepared for this site. The critical cross section is cut through one of the steepest portions of the coastal bluff and area of the proposed home site to estimate for potential of bluff failures. This information also corroborated the slope stability portion of the estimated 50 year future coastal bluff recession setback.

Specifically we did the following:

A. Document review of information provided by property owner in our files pertinent to the site and region, including:

- Coastal Bluff Recession Map and Cross Sections, prepared by Haro, Kasunich and Associates Inc., dated 9 March 2015.
- Architectural plan sheets A3.1 – A3.4, prepared by Pearson Design Group, Undated.
- Site sections L4.1 – L4.3, prepared by Verde Design, dated 28 July 2015.
- Civil plan sheets C1.0 – C8.0 dated 5 August 2016, prepared by Mesiti-Miller Engineering.
- Conferred with and discussed scope and requirements with San Mateo County Public Works department, Diana Shu and County Geologist Jean Demouthe.

B. Met at the site with the Project Biologist to review environmental constraints at the project site. HKA placed wood stakes at each of the test boring locations and contacted USA Underground as mandated by law within 48 hours of scheduled drill date.

C. HKA obtained a geotechnical soil boring permit for this project site from San Mateo County Health Services Division (SMCHSD). A soil boring location map and

Drilling Notification Form was submitted to SMCHSD a minimum 48 hours prior to the scheduled drill date.

D. Subsurface exploration consisting of logging and interval sampling of soils encountered in five (5) exploratory test bore holes advanced in the area of the proposed residences between 13.5 to 25.0 feet deep. The exploratory test bore holes were advanced using portable drilling equipment that was hand carried onto the site in parts and built in place of each test boring location. The soil samples obtained were sealed and returned to our laboratory for testing. After completion of each test boring the drilled shaft was infilled with a cement grout mixture specified by SMCHSD. The mixing and placement of the grout was performed by a drilling contractor with a C-57 license. Pictures were taken during this operation and forwarded to Lawlor Land Use see Appendix F of this report. Soil cuttings were hauled off site for proper disposal.

E. A total of five (5) percolation test holes were advanced near the areas of the proposed drain fields between 2.15 to 4.04 feet deep. The holes were advanced using hand auger gear. Percolation tests were performed within the drilled shafts following an EPA and San Mateo County procedures for determining percolation test rate within soils. The holes were backfilled with soil cuttings.

F. Laboratory testing of select samples obtained. Moisture content and dry density tests of selected samples was performed to evaluate the consistency of the in situ soils. Soil strength parameters were derived from in-situ field penetration tests (SPT), an unconfined compression test, and a laboratory direct shear test on select samples under saturated and in-situ moisture contents. Atterberg limits tests were performed on select clay soil samples to qualify its expansion potential. Corrosion testing was performed on bulk samples of site soil.

G. Quantitative slope stability analysis was performed on a critical cross section (Section 3) cut through on one of the steepest portions of the coastal bluff and the area where one of the proposed home sites is nearest the bluff. The location of the critical cross section was selected by HKA from site observations and review of a topographic map prepared for this site. The analysis were run under static and pseudo static (seismic) loading conditions.

H. Geotechnical analysis with consideration to our laboratory test results, slope stability results, the estimated 50-year future coastal bluff recession setback, our experience in the area, and engineering judgement. Our analysis developed geotechnical design parameters for building foundations, grouted soil anchors, retaining walls, and concrete slab-on-ground.

l. Preparation of this report summarizing our findings, conclusions, and recommendations.

Site Location and Project Description

The project site consists of five privately owned parcels on the seaward side of California State Highway 1. The parcels make up an undeveloped field bound by Vallemar Street to the east and Juliana Avenue to the south. The project site is a relatively flat, elevated marine terrace that sits approximately 45 feet above sea level. The project site is covered with grasses, ice plant, trees, and shrubs. There is evidence of historical grading, associated with the construction of Vallemar Street and possibly Highway 1. A fill wedge descends from the seaward side of Vallemar Street down to the landward side of the project site. The coastal bluffs that line the seaward side of the project site are steeply cut from years of wave attack. A drainage swale which appears to be part of the construction of Juliana Avenue runs along the southeast side of the project side and discharges into a small ravine descending to the beach below.

Based on interaction with Lawlor LandUse and review of conceptual drawings, HKA understands the proposed improvements consists of the following. Grading of building pads on each site by means of cut and fill construction methods. A new single family dwelling and driveway is shown to be constructed on each parcel. The parcels are designated by lot lettering. Lot A on the north corner of the site and Lot G to its east. Lot

D, E, and F are located on the southeast side of the project site adjacent to Juliana Avenue. Lot D is the seaward most, Lot F Is adjacent to Vallemar Street, and Lot E is between the two. The new homes are referred to as the "Home A, D, E, F, and G" respective to the lot letter for this project and within this report.

Home A and Home G are shown to have frontage onto Vallemar Street. The driveway and parking area for Home A is shown to be built up with 2 to 5 feet of engineered fill and the 2 story house is shown to be cut into the fill embankment with its upper level even with the parking area. Home G is shown to be cut into the site up to five feet and fill placed on the down slope side of the access driveway. Retaining or basement walls are anticipated on the upslope side of the Home A and Home G.

Home D, Home E, and Home F are shown to have frontage onto Juliana Avenue. Each of these home will require construction of a small bridge to cross the drainage swale adjacent Juliana Street. Starting at home F, a cut is shown on the upslope side on the order of 2 to 4 feet and a fill on the order of 4 feet shown the downslope side. Home E is shown to have significantly less grading with cut and fills on the order of 1 to 3 feet. Home D is shown to have minimal grading with cut and fills 1 foot thick or less.

Field Exploration

On 22 and 24 March 2016 a total of five (5) exploratory test bore holes were advanced at the project site. One test boring near the location of each home site. On 30 March 2016 we returned to the site and advanced five (5) percolation test bore holes near the area of the proposed drain fields for the homes.

The exploratory test boring in the area of the new homes were advanced to depths of 13.5 to 25 feet below the ground surface (bgs). The exploratory test bore holes were advanced using solid flight auger portable drilling equipment that was hand carried onto the site in parts and built at the locations of each test borings. The soil samples obtained were sealed and returned to our laboratory for testing. After completion of each test boring the drilled shaft was infilled with a cement grout mixture specified by SMCHSD. The mixing and placement of the grout was performed by a drilling contractor with a C-57 license. Pictures were taken during this operation and forwarded to the Lawlor Land Use. Soil cuttings were placed into 5 gallon buckets and hand carried to a waiting truck. The packed soil was transferred off site for proper disposal See Appendix F of this report for pictures of drilling operation.

A total of five (5) percolation test holes were advanced near the areas of the proposed drain fields between 2.15 to 4.04 feet deep. The holes were advanced using hand auger gear. Percolation tests were performed within the drilled shafts following an EPA procedure for determining percolation test rate within soils. The holes were backfilled with soil cuttings.

To provide extra protection against disturbance of the site a large tarp was laid over the work areas. Each exploratory and percolation test boring was advanced through the tarp and all buckets, drilling parts, soils cuttings etc. were carefully handled over the tarp. After the work area was cleared the tarp was folded up and the plants were replaced.

In-situ samples were collected from within the exploratory test borings. Samples were obtained by driving a California Sampler (3 inch outside diameter) or split spoon sampler (2 inch outside diameter) up to18 inches in depth at select elevations using a standard 140-pound hammer over a 30-inch drop. The amount of blows to drive the sampler 1 foot were recorded and presented on our logs of borings attached to this letter (Figures 4 to 13). The logs also include profiles of the percolation test bore holes.

The approximate location of test bore holes are shown on our Test Boring Location Map (Figure 2). The soil encountered in the borings was continuously logged in the field and described in accordance with the Unified Soil Classification System (ASTM D2487).

Laboratory Testing

The laboratory testing program was directed toward determining pertinent soil engineering, corrosion, and index properties.

The natural moisture content was determined on select samples and is recorded on the Logs of Test Borings at the appropriate depths. Since water has a significant influence on soil, the natural moisture content provides a rough indicator of the soil's compressibility, strength, and potential expansion characteristics.

A saturated moisture direct shear tests and an in-situ moisture unconfined compression test were completed to determine strength properties for coastal terrace. Density tests were also performed to aid in the assignment of soil properties to each soil type.

Atterberg limits tests were performed on select clay soil samples to qualify its expansion potential. The Atterberg limits were run on a select near surface sample collected from within the exploratory test boring advanced on each of the home sites.

The results of the field and laboratory testing appear on the "Logs of Test Borings" opposite the samples tested (Figures 4 through 13).

Subsurface Conditions

In general within test bore holes advanced in the area of the home sites the soil profile encountered consisted of clay soil over either silty sand, clayey sand, sand with silt or a combination of thereof all overlying a hard bedrock formation. The overburden soils are interpreted as coastal terrace and were loose near the surface and became mostly medium dense and occasionally dense with depth. The coastal terrace was mixed with organics and roots within the upper 1 to 2 feet as a result of top soil development. The upper 6 to 8 feet of the coastal terrace was stiff to very stiff sandy clay or clay. In the area of Home G the clay layer was 15 feet thick. The silt clay sand mixtures extended below the clay layer to a depth of approximately 25 feet bgs where drilling refusal was encountered. We interpret this contact as hard bedrock.

In the percolation test bore holes advanced in the area of the proposed drain fields for Home A, Home F, and Home G consisted of top soil in the top foot and silty sand below that. These locations had low to moderately low percolation rates. In the locations of the drain fields for Home D and Home E the soil encountered consisted of 1 foot of top soil over clay. These locations had zero percolation.

Expansive Soils

Based on the measured Atterberg Limits, the clay soil collected within the foundation zone of the home sites was qualified to have moderately high potential for expansion and in the upper 2 to 3 feet at Home D and Home E it has a high potential for expansion. There was a large standing puddle observed in this area for several weeks during the course of our field exploration phase of this study. The clay soil with moderately high potential for expansion (Homes A, F, & G) can be mitigated for foundation support if the recommendations in this report are carefully followed during development of project plans and during construction. The clay soil with high potential for expansion in foundation zones (Homes D & E) should be removed and replaced with select granular fill.

Groundwater

Groundwater was encountered within our test bore holes advanced at Homes D, E, & F adjacent to Juliana Avenue. The groundwater was encountered at 17 feet bgs near Vallemar Street and 13 feet at Home D closer to the bluff. The groundwater appears to be perched upon the bedrock and seeping through the terrace near the contact. That being said saturated soils and active seeps in the coastal terrace soils should be anticipated and planned for by designers and contractors. Retaining wall back drains and under slab blanket drains will be essential for the design of this structure. It is recommended to relieve drainage collected in these subsurface systems through a gravity flow if possible.

Liquefaction Potential

Liquefaction is a phenomenon where loose to medium dense soils with low to zero cohesion that are submerged and subject to seismic shaking can temporarily lose their shear strength. This is most common in young alluvial soils near sloughs, rivers, and flood plains. Although medium dense sand and silt was encountered just above the bedrock formation it was relatively thin and more than 15 feet bgs. Based on the lack of evidence of ground effects related to liquefaction occurring within coastal terrace the potential for liquefaction at the site is low.

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	200
Soil 3	135	45	1000

Erosion

Surficial soils at the site are prone to erosion which can be severe where there are steep slopes and uncontrolled runoff, particularly where the natural drainage is modified by the works of man and not properly controlled. Typically, once the upper surface of the material is breached by a rill or a gully, erosion proceeds at an accelerated rate, and the rills and gullies deepen and migrate headward (upslope). This process may contribute to the initiation of debris flows if rills and gullies are not mitigated or maintained and if surface drainage controls are not adequately designed and constructed.

Surface Drainage

The project site is located near a coastal bluff comprised partially of coastal terrace deposits that are susceptible to erosion, particularly by concentrated uncontrolled runoff of surface drainage. The proposed improvements will increase the runoff flow rate shedding away from the site. Collection of surface runoff into drain lines with single discharge points will further concentrate it relative to the sheet flow type drainage prior to improving the site. Development of an engineered drainage plan that conveys surface runoff to multiple discharge locations and promotes sheet type flow of collected drainage is recommended for this site. Level drainage spreaders are an example of this type of system.

Geotechnical Related Seismicity

The improvements should be designed in conformance with the most current California Building Code (2013 CBC). For seismic design, the soil properties at the site are classified as **Site Class "D"** based on definitions presented in Table 1613.5.2 in the 2013 CBC. The longitude and latitude were determined using a satellite image generated by Google Earth. These coordinates were taken from the approximate middle of the area of the proposed improvements:

Longitude = -122.5169, Latitude = 37.5300

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= 2.269 g

SM_1= 1.439 g

SD_s= 1.512 g

SD_1= 0.960 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 0.89 g and the site coefficient F_{PGA} for Site Class D is 1.0. The MCE_G peak ground acceleration adjusted for Site Class effects is $PGA_M = F_{PGA} * PGA$

$$PGA_M = 1.0 * 0.89\ g = 0.89\ g$$

Quantitative Slope Stability Analysis

Stability analysis was performed on the worst case or critical cross section cut through the coastal bluff and Lot D. 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 cross section is included with this report (Appendix C). 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).

Circular failure surfaces were assumed for this model. GSTABL7 program uses the Simplified Bishop Method of Slices 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. The computer program assumes many failure surfaces using initiation and termination points on the ground surface selected by the user. These chosen points represent the toe and scarp of each potential landslide in relation to the assumed failure surfaces. The critical trial failure surface from the pseudo static analysis condition was selected as the projected failure surface in the development of design parameters.

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 0.89g. 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 0.51g.

Geometric Assumptions

For our analysis, failure surfaces were 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. The trial failure surfaces used in the analysis were selected using engineering judgment as well as the software's ability to generate many random surfaces.

Slope Stability Conclusions

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 B of this report.

Table 2: Slope Stability Analysis Results

Bluff Recession Section	Loading Condition	Minimum Factor of Safety Against Sliding	Meet or Exceed Required FS
3	Static	2.48	Yes
3	Pseudo Static	1.27	Yes

Limitations of Analysis

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

Percolation Testing Set-Up

HKA performed percolation tests near the proposed drain field areas. The exact areas of the proposed drain fields could not be tested due to presence of trees, bushes, and other sensitive plants. Based on interaction with the project Civil Engineer Mesiti Miller Engineering (MME), HKA understand the drain fields will consist of pits on the order of 2 to 4 feet deep infilled with drain rock. Depths and locations of the percolation test holes were also worked out with MME.

On 30 March 2016 five (5) percolation test borings were drilled using a 6 inch diameter hand auger. After drilling to the selected depth, a layer of 1/4" angular gravel approximately 2 to 3 inches thick was placed at the bottom of each percolation test boring. Three (3) inch diameter NDS pipe was prepared for each test hole by cutting slots with a hacksaw along the bottom 6 inches of the pipe sections. The slotted pipe sections were placed in each test hole with additional gravel placed between the pipe and the borehole sidewall to secure the pipe in place. The bottom 12 inches of the test bore holes were filled with clear water 4 times within 24 hours prior to commencing the percolation test.

Percolation Testing

On 31 March 2016, HKA returned to the site after the 24 hour soaking period to conduct the percolation tests.

The percolation test holes were inspected and all but P-4 and P-5 were completely drained. Percolation test holes P-4 (Lot 4) and P-5 (Lot 5) still had 12 inches of standing water in the bottom of the test holes. These percolation test were advanced into a layer of clay soil and near the standing puddle.

On the same day the 6-inch falling head percolation tests were performed as follows:

- Clear water was placed within the bottom 6 inches of each test hole.
- A water level reading was taken every 30 minutes and the percolation test hole was refilled with clear water to 6 inches above the bottom of the hole.
- Up to eight (8) water level readings were taken in each percolation test hole. If 3 consecutive readings were within 1/16 of an inch of each other the EPA test method recommends to stop the test.
- The percolation rate in inches per hour was calculated by dividing the change in height of the water level in inches by the interval between readings in hours.
- The last change in the water level reading and consideration to the set of readings was used to report the percolation rate for the respective test hole.

Percolation Test Results

Test Hole ID	Percolation Zone (feet)	Percolation Rate (in/hr)
P-1	1.146 to 2.146	3.9
P-2	1.146 to 2.146	1.4
P-3	3.042 to 4.042	0.6
P-4	1.958 to 2.958	0.0
P-5	1.958 to 2.958	0.0

Graphical and tabulated results are included in the Appendix D of this report.

DISCUSSION, CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our investigation, the proposed improvements at the referenced site will be subject to "ordinary risks", as defined in the "Scale of Acceptable Risks From Geologic Hazards" in Appendix D of this report provided the design criteria and recommendations presented in this report as well as those in our geologic report for this project are incorporated into the design and construction of the proposed project and maintained for the life of the development.

The primary geotechnical considerations at the site include strong seismic shaking, adequate foundation support of new buildings, temporary cut slopes during construction, expansive clay soil in foundation zone, subsurface seepage, coastal bluff erosion, and control of concentrated surface runoff,.

Based on our analysis of site soils conditions and consideration to the recommended 50 year future coastal bluff recession setback, the proposed Home A, Home B, and Home F should be supported by conventional spread foundations embedded into an earthen mat of moisture conditioned on-site soils prepared in accordance with this report. Home D and Home E should be supported by conventional spread foundations embedded into an earthen mat of select granular engineered fill prepared in accordance with this report.

Groundwater was encountered within our test bore holes at the time they were drilled. Saturated soils and active seeps in the coastal terrace soils should be anticipated and planned for by designers and contractors. Retaining wall back drains and under slab blanket drains will be essential for the design of these structures. It is recommended to relieve drainage collected in these subsurface systems through perforated collection pipes tied to solid drain lines that are conveyed to a discharge location by gravity flow if possible.

Based on our interaction with the project design team HKA understands a partial basement is proposed below Home A and Home G. The basement is not shown along the seaward perimeter of these homes. Excavations for the partial basements are shown to create cut slopes on the order of 5 to 10 feet tall into coastal terrace deposits. The cut slopes should be laid back to safe slope gradients or temporary cantilever or tied back shoring utilizing top down construction methods should be employed. As an alternative the basement wall could be constructed and then backfilled. Once this decision is made HKA should be consulted to make supplement recommendations as needed that are compatible with the project goals. The pier criteria can be used for the basement wall or for Home D and Home E as an alternative design. The architect, civil engineer, and structural designer should assume wet to saturated coastal terrace will be encountered within the cut face of the excavation. Criteria for drilled shaft, grouted, post tensioned soil anchors are included for use in the temporary shoring system or permanent basement retaining walls if needed.

The following recommendations should be used as guidelines for preparing project plans and specifications, and assume that **Haro, Kasunich & Associates** will 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.

General Site Grading

1. The geotechnical engineer should be notified **at least four (4) working days prior to any grading or excavating foundations** so the work in the field can be coordinated with the grading contractor and arrangements for testing and observation can be made. The recommendations of this report are based on the assumption that a representative from HKA will perform the required testing and observation during grading and construction. It is the owner's responsibility to make the necessary arrangements for these required services.

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

3. Areas to be graded or to receive proposed improvements should be cleared of all obstructions and fill materials, including trees not designated to remain and other unsuitable material. Existing depressions or voids created during site clearing should be backfilled with engineered fill. Any surface or subsurface obstructions, or questionable material encountered during grading, should be brought immediately to our attention for proper exposure, removal, and processing as directed.

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

5. On-site soils reused as engineered fill and imported select granular fill should be placed in thin lifts not exceeding 8 inches in loose thickness. On-site clay soil approved for re-use by HKA should be, water conditioned to a moisture content about 3 to 6 percent above optimum, and compacted to 87 to 89 percent relative compaction back up to the ground surface. Imported select granular fill should be, water conditioned to a moisture content about 2 to 4 percent above optimum, and compacted to at least 90 percent

relative compaction. The upper 8 inches of subgrade should be compacted to at least 95 percent relative compaction. Aggregate base below pavements should likewise be compacted to at least 95 percent relative compaction.

6. If grading is performed during or shortly after the rainy season, the grading contractor may encounter compaction difficulty with the wet soils. If compaction cannot be achieved after adjusting the soil moisture content, it may be necessary to use imported fill or gravel and stabilize the bottom of the excavation with stabilization fabric.

7. Provided they can be adequately moisture conditioned (or dried back) prior to use, the on-site soils appear generally suitable for use as engineered fill, however clay soils with intermediate or high plasticity may be unsuitable. Materials used for engineered fill which must be imported should be free of organic and deleterious material, contain no rocks or clods over 4 inches in dimension, and should contain no more than 15 percent by weight of rocks larger than 2½ inches. Imported fill should also be granular, have a Plasticity Index of less than 18, and should have sufficient binder to allow excavations to stand without caving. Prior to delivery to the site, a representative sample of proposed import should be sent to our laboratory for evaluation.

8. We estimate shrinkage factors of about 17 percent for the on-site materials when used in engineered fills.

Cut and Fill Slopes

9. Temporary excavations should be properly shored and braced during construction to prevent sloughing and caving at sidewalls. The contractor should be aware of all CAL OSHA and local safety requirements and codes dealing with excavations and trenches.

10. The excavation along the northeast side of Home A and Home G is shown to be on the order of 5 to 10 feet deep bgs. Designers should assume the cut slope to be comprised of stiff or medium dense coastal terrace deposit.

11. It should be anticipated that perched ground water will be actively seeping from the face of the cut slope excavated into the coastal terrace deposits. The thickness of the seepage layer will depend upon the time of year the excavation is made. Designers and contractors should plan accordingly.

12. Temporary cut slopes excavated into the coastal terrace deposits should be inclined at a slope gradient of 1:1 (H:V) or flatter where no seepage is observed from face of cut slope and 2:1 (H:V) or flatter where seepage is observed. Depending on the amount of seepage from the face of the cut slope shoring may be required. Temporary cut slopes excavated for the project are considered those that are to remain from 24 hours up to the start of the rain season.

13. For design of lateral earth support systems used for temporary shoring or permanent retaining walls a lateral earth pressure equivalent to a fluid weighing (EFW) 40 pcf should be used under drained conditions (i.e. gravel drain) for an active condition. For at rest or restrained condition use 30H psf uniform load. For 2:1 back slope gradients add 15 pcf and 10H psf respectively. If the shoring is to be designed without a drain or "undrained condition" Add 45 pcf or 30H respectively.

14. Compacted fill slopes should be constructed at a slope inclination not steeper than 2:1 (horizontal to vertical) at 90 percent relative compaction. Fill slopes with these recommended gradients may require periodic maintenance to remove minor soil sloughing. All fills must be adequately benched into firm native soil, and keys for stability will be required at the toe of the fill slope. The toe key should be at least 8 feet wide and should extend at least 2 feet into firm native soil. The bottom of the toe key should be sloped downward at about 2 percent toward the back of the key.

15. There should be a minimum of 10 feet horizontal separation between the top of supporting soil that will be used for skin friction and the face of slope.

16. In order to maintain stable slopes at the recommended gradients, it is important that seepage forces and accompanying hydrostatic pressure be relieved by adequate drainage. Adequate backdrains in keyways and benches should be provided as

determined necessary by HKA. The locations of backdrains and outlets would be determined by a representative of HKA in the field during grading.

17. Following grading, exposed soil should be planted as soon as possible with erosion-resistant vegetation.

18. After the earthwork operations have been completed and HKA has made the required observations of the work, no further earthwork operations shall be performed without the direct observation of HKA.

Conventional Spread Foundations

19. The new homes should be supported by conventional spread foundations that are embedded into an earthen mat of engineered fill that extends a minimum of 18 inches below bottom of foundations elements and 6 horizontal feet beyond the outer most edge of foundation. For Home A, B, and F the earthen mat should be comprised of re-used on-site soils. For Home D and Home E the earthen mat should be comprised of select import granular fill.

20. Foundations should be embedded into an earthen mat of engineered fill a minimum 2 feet deep. Actual footing depths should be determined in accordance with anticipated use and applicable design standards. Conventional footings should be reinforced as

required by the structural designer based on the actual loads transmitted to the foundation.

21. Foundations designed in accordance with the above may be designed for an allowable soil bearing pressure of 1,700 psf for dead plus live loads. The allowable bearing capacity may be increased by one-third to include short-term seismic and wind loads.

22. Deep foundation elements (piers) may be used as an alternative foundation. See the section titled “Skin Friction Pier Foundations”.

23. Lateral load resistance for structures supported on spread footings may be developed in friction between the foundation bottom and the supporting subgrade. A friction coefficient of 0.30 is considered applicable.

24. Footings located adjacent to other footings or utility trenches should have their bearing surfaces founded below an imaginary 2:1 plane projected upward from the bottom edge of the adjacent footings or utility trenches.

25. Total and differential settlements across the new homes are anticipated to be less than 1 inch.

26. All footing excavations should be thoroughly cleaned and observed by HKA 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 that the footings are in accordance with our recommendations

Skin Friction Pier Foundation

27. Drilled pier foundations can be used where the structural designer determines deep foundation are necessary to resist lateral overturning forces, concentrated axial loads, or simply as an alternative to the earthen mat construction. Drilled piers may also be considered for foundation support of temporary shoring to support cut slopes.

28. Actual pier depth will depend upon a force analysis by the project design professional; however the piers should have a minimum diameter of 18 inches and minimum spacing of 4 feet on center. The piers should be embedded into the coastal terrace a minimum of 10 feet. Based on our recent exploratory boring we were able to drill through the coastal terrace and hit drill refusal on the bedrock at a depth of 25 feet bgs. It should be noted we drilled with a 4 inch diameter solid flight auger portable drill rig.

29. The upper 3 foot of the pier should be neglected for passive resistance and skin friction. The piers should be designed to withstand an uplift pressure of 450 psf in the upper 4 feet.

30. For vertical bearing capacity in the upper 10 feet of the pier an allowable skin friction of 600 psf should be applied to the surface of the pier below the neglect depth. For resistance to uplift forces, an allowable skin friction of 300 psf should be applied to the surface of the pier below the neglect depth. For each additional foot of pier depth below 10 feet deep add 15 psf of skin friction for vertical bearing and 7.5 psf to resist uplift. Maximum allowable skin friction is 700 psf for vertical being and 350 for uplift resistance. The increased value should be applied to the full depth of the pier.

31. A passive lateral earth pressure with an equivalent fluid weight of 300 pcf acting over 2.0 pier diameters should be applied to the pier below the depth of neglect.

32. Reinforcing vertical steel for the concrete piers should extend the full depth of the excavation to a point 3 inches above the bottom of the pier hole.

Perched Groundwater Drainage

33. Seepage should be collected from behind retaining walls and beneath slabs-on-ground in gravel drains with perforated pipe. The collected drainage is recommended if possible to be relieved by gravity flow to a discharge area approved by a representative from HKA. It is imperative to waterproof the exterior basement retaining walls and floor slab of the new homes to protect against moisture intrusion from perched groundwater seepage.

34. The drainage systems should be a minimum 12 inches wide behind walls or 12 inches deep beneath slabs and comprised of Class 1 Type A gravel with a perforated pipe placed near the bottom of the drain a thin bedding of gravel.

35. A representative from HKA should observe the drainage system just after the pipes have been placed over the gravel bedding.

Concrete Slab-On-Ground

36. Concrete slab floors should be constructed with an under slab drain comprised of a 12 inch thick blanket of gravel that has been set with a vibratory plate. The bottom of the subexcavation should be scarified, moisture conditioned or dried back as needed, and compacted to a minimum 95 percent.

37. For construction of the under slab drain use¾ inch nominal drain rock (or equivalent) wrapped in filter fabric. Furthermore, a 14 mil vapor barrier should be placed below the slab and wrapped under the footings and up to the side of exterior of the building. A perforated pipe should be embedded into the blanket drain that conveys collected drainage by gravity flow to a discharge location seaward from improvements, but preferably landward of the coastal bluff setback. Drainage from basement retaining wall back drains or surface drainage should not be allowed to enter into the under slab blanket drain.

38. To reduce the potential for cracking and curling as well as other undesirable defects the concrete slab-on-grade design, placement, and curing should be done in accordance with the most recent version of ACI 302.1R-04.

39. If floor wetness would be unacceptable for the buildings for reasons such as moisture sensitive floor covering or interior humidity control a vapor barrier should be placed below the slab. Vapor barriers should be overlapped a minimum of 6 inches at the joints and carefully fitted around service openings.

40. Whether to locate the vapor barrier in direct contact with the slab or beneath a blotter layer of granular fill should be made with careful considerations to many factors directly and indirectly related to concrete construction. Such factors include but are not limited to; whether a water tight roof membrane is in place prior to slab construction, sequence of slab construction in relation to other construction activities requiring water, and the floor covering manufacturer's recommendations. Proposed installation should be independently evaluated as to the moisture-related sensitivity of subsequent floor finishes, project conditions, schedule, and the potential effects of slab curling and cracking. We also recommend that a qualified experienced waterproofing specialist be included on the design team and these recommendations and any revised or supplemental recommendations they make be included in the final design construction documents and implemented during construction.

41. If placement of concrete in direct contact with the vapor barrier is selected measures to minimize potential for shrinkage related defects such as but not limited to slab curling, dominant joints, and plastic or drying shrinkage cracking will be required. Measures would include selection of concrete mixtures with low potential for shrinkage and/or tighter joint spacing.

42. If a blotter layer of granular fill over the vapor barrier is selected it should be a minimum of 4 inches thick, trimmable, and compactible at low moisture content (4 to 5 percent). The use of cushion or clean sand with uniform particle size is not recommended for use as a blotter layer of granular fill. Crusher run material graded from 3/4 inch down to rock dust is suitable. The blotter layer of granular fill should be compacted to a minimum 95 percent relative compaction in accordance with ASTM D1557 To prevent the granular fill from becoming a water reservoir (contributing to floor wetness) it will be imperative to keep it dry after preparation has been completed.

Retaining Walls and Lateral Pressures

43. For design of retaining walls up to 20 feet in height and fully drained, the following design criteria may be used:

A. Active earth pressure for walls allowed to yield (up to ½ percent of their height) is that exerted by an equivalent fluid weighing 40 pcf for a level back slope gradient; 55 pcf for a 2:1 backslope gradient, and 48 pcf for backslope gradients between 3:1 and 6:1. This is assuming a fully drained condition. For un-drained conditions add an additional 40 pcf to the respective active earth pressure.

B. Where walls are restrained from moving at the top, design for uniform wall pressure of 30H psf/ft for level backfill and 40H psf/ft for 2:1 backfill slope gradient, and 36H psf/ft for backslope gradients between 3:1 and 6:1 where H is the height of the wall. This is assuming a fully drained condition. For un-drained conditions add an additional 30H psf/ft to the respective active earth pressure.

C. Site retaining walls should be supported by conventional spread footings embedded into firm coastal terrace or pier and grade beam foundations. The foundations should be designed and constructed in accordance with the recommendations of this report.

D. For seismic design of critical structures, a nominal earthquake load equal to 10 H^2 lbs/horizontal foot of wall may be assumed to act at 0.6H above the heel of the wall base (where H is the height of the wall).

E. In addition, the walls should be designed for a surcharge loads for adjacent live or dead surcharge loads which will exert a force on the wall. Contact HKA for a detailed evaluation of lateral surcharge loads acting against retaining walls.

F. For fully drained conditions as delineated above, we recommend that permeable material meeting the State of California Standard Specifications, Section 68-1.025, Class 1, Type A, or an approved equivalent be placed behind the wall, with a minimum continuous width of 12 inches, and extend the full height of the wall to within 1-foot of the ground surface. A 4-inch diameter perforated drain pipe (with perforations placed downward) should be installed within 4 inches of the bottom of the granular backfill and be discharged to a suitable location. Surface drainage should not be allowed to enter retaining back drains, nor should back drains be tied to under slab blanket drains.

G. Wall backfill should be compacted to a minimum of 90 percent relative compaction. The backfill material should be approved by HKA

H. HKA should observe foundation excavations during to observe anticipated soil conditions and excavation depths.

<u>Retaining Wall Tie Backs</u>

44. Where the structural designer deems tie backs necessary, drilled shaft and grouted tie backs should be used in conjunction with the selected foundation system for the retaining wall.

45. Tie backs should be constructed out of steel reinforcement that extends the entire length of the tie back and concrete grout in the bonded zone (stressing zone). Tie backs

should be designed and constructed, and tested in accordance with the Post Tensioning Manual by the Post Tensioning Institute.

46. Tie backs should be a minimum 6 to 8 inches in diameter. If larger diameter tie backs are needed HKA should be notified to make appropriate adjustments to the recommendations.

47. Tie backs should be a minimum 30 feet in length and installed between 20 to 30 degrees below an imaginary level horizontal line.

48. Tie backs should have a minimum un-bonded length of 15 feet and minimum bonded (stressing) length of 15 feet.

49. The structural designer should use a bond stress between the surface area of the grouted portion of the tie back and the drilled shaft. For non-pressure grouting applications a bond stress of 2000 psf should be applied and for pressure grouting applications 3000 psf should be applied.

50. The bonding strata is fine to medium sands and gravels. This will require either a cased drilled shaft or hollow stem drill augers to keep the shaft from collapsing in the bonded length of the tie back.

Utility Trenches

51. Trenches must be properly shored and braced during construction or laid back at an appropriate angle to prevent sloughing and caving at sidewalls. The project plans and specifications should direct the attention of the contractor to all CAL OSHA and local safety requirements and codes dealing with excavations and trenches.

52. Utility trenches that are parallel to the sides of buildings should be placed so that they do not extend below an imaginary line sloping down and away at a 2:1 (horizontal to vertical) slope from the bottom outside edge of footing elements. The structural design professional should coordinate this requirement with the utility layout plans for the project

53. Trenches should be backfilled with granular-type material and uniformly compacted by mechanical means to the relative compaction as required by county specifications, but not less than 95 percent under paved areas and 90 percent elsewhere. The relative compaction will be based on the maximum dry density obtained from a laboratory compaction curve run in accordance with ASTM Procedure #D1557.

54. We strongly recommend placing a three-foot (3') concrete plug in each trench where it passes under the exterior foundations. Care should be taken not to damage utility lines.

55. Trenches should be capped with 1.5 feet of relatively impermeable soil.

Surface Drainage

56. An engineered drainage plan to handle surface runoff should be developed for this site. Site drainage should be adequately controlled both during and after construction.

57. Surface runoff should be collected into level spreaders to result in sheet flow type discharge.

58. The collected runoff should be discharged in at least two locations to minimize impact. The specific discharge locations should be selected by the engineer who prepares the drainage. As an alternative a single level spreader can be used that promotes sheet type flow. The level spreaders should be located as from the bluff edge as possible. Landward of the 50 year future coastal bluff recession setback is recommended.

59. On-site retention is not recommended within the 50 year future coastal bluff recession setback.

60. All exposed soil should be landscaped and permanently protected against erosion as soon as possible after grading.

61. We recommend that full gutters be used along all roof down eaves to collect storm runoff water and channel it through closed rigid conduits to a suitable discharge point a minimum 10 feet away from all structural improvements.

62. Surface runoff should **not** be allowed to flow onto graded or natural slopes with gradients equal to or steeper than 3:1 (H:V). Consideration should be given to catch basins, berms, concrete v-ditches, or drainage swales at the top of all slopes to intercept runoff and direct it to a suitable discharge point.

63. Surface drainage should include provisions for positive gradients so that surface runoff is not permitted to pond adjacent to foundations and on pavements. Surface drainage should be directed away from the building foundations, at a minimum gradient of 2 percent for a distance of at least 10 feet to an adequate discharge point. Concentrations of surface water runoff should be handled by providing necessary structures, such as paved ditches, catch basins, etc.

64. Irrigation activities at the site should be done in a controlled and reasonable manner. Planter areas should not be sited adjacent to walls; otherwise, measures should be implemented to contain irrigation water and prevent it from seeping into walls and under foundations.

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

66. Drainage patterns approved at the time of fine grading should be maintained throughout the life of proposed structures.

Curtain Drain

67. Groundwater seeping through the terrace deposits perched upon the bedrock formation was encountered at this site. Pervious pavements are also proposed adjacent to some of the new homes. To protect the homes from moisture intrusion through seepage curtain drains should be constructed on the upslope side of the homes extending beyond the footprint a minimum 10 horizontal feet. The curtain drains should also wrap around the sides of the homes where pervious pavement is placed adjacent to the building. Where basement retaining walls with back drains are proposed such as Home A the curtain drain is not required.

68. The curtain drains should be a minimum 12 inches wide and extend to a minimum depth of 18 inches below bottom of foundation elements. The curtain drains should be placed within 3 horizontal feet of the outer most edge of the building foundation. For this project we anticipate curtain drains to be on the order of 4 to 6 feet deep.

69. The trench for the curtain drain should be lined on the side adjacent to the home with a vapor barrier, a 4 inch diameter perforated pipe with holes placed down should be set on a thin bed of gravel along the bottom of the drain, and the trench infilled with drain rock wrapped in filter fabric. The perforated pipe should be connected to a solid drain pipe that conveys the collected drainage away from the trench and discharges it into a level spreader down slope from the home.

Pavement Design

70. R-Value tests have not been performed for this project.

71. To have the selected pavement sections perform to their greatest efficiency, it is very important that the following items be considered:

a. Scarify and moisture condition the top 8 inches of subgrade and compact to a minimum relative compaction of 95 percent, at a moisture content which is about 4 percent above laboratory optimum value.

b. Provide sufficient gradient to prevent ponding of water.

c. Use only quality materials of the type and thickness (minimum) specified. All baserock (R=78 minimum) must meet CALTRANS Standard Specifications for Class 2 Untreated Aggregate Base

(Section 26). All subbase (R=50 minimum) must meet CALTRANS Standard Specifications for Class 2 Untreated Aggregate Subbase, (Section 25). Angular gravel (ASTM D448) or Class II permeable aggregate base (Caltrans Spec) should be used below pervious pavements.

d. Compact the baserock and subbase uniformly to a minimum relative compaction of 95 percent. Gravel or permeable aggregate baserock should be placed in 8 inch lifts and set using a vibratory plate under observation of HKA.

e. Place the asphaltic concrete only during periods of fair weather when the free air temperature is within prescribed limits.

f. Maintenance should be undertaken on a routine basis.

Plan Review, Construction Observation and Testing

72. Our firm should be provided the opportunity for a general review of the project plans prior to construction so that our geotechnical recommendations may be properly interpreted and implemented. The purpose is to determine if this preliminary report is adequate and complete for the final planned grading and construction. It is not intended that the geotechnical engineer approve or disapprove the plans, but to provide an opportunity to update the preliminary report and include additions or qualifications as

necessary. If our firm is not accorded the opportunity of making the recommended review, we can assume no responsibility for misinterpretation of our recommendations.

73. We recommend that our office review the project plans prior to submittal to public agencies, to expedite project review. The recommendations presented in this report require our review of final plans and specifications prior to construction and upon our observation and, where necessary, testing of the earthwork and foundation excavations. Observation of grading and foundation excavations allows anticipated soil conditions to be correlated to those actually encountered in the field during construction.

IMITATIONS AND UNIFORMITY OF CONDITIONS

1. The conclusions and recommendations noted in this report are based on probability and in no way imply that the proposed improvements will not possibly be subjected to ground failure or seismic shaking so intense they will be severely damaged or destroyed.

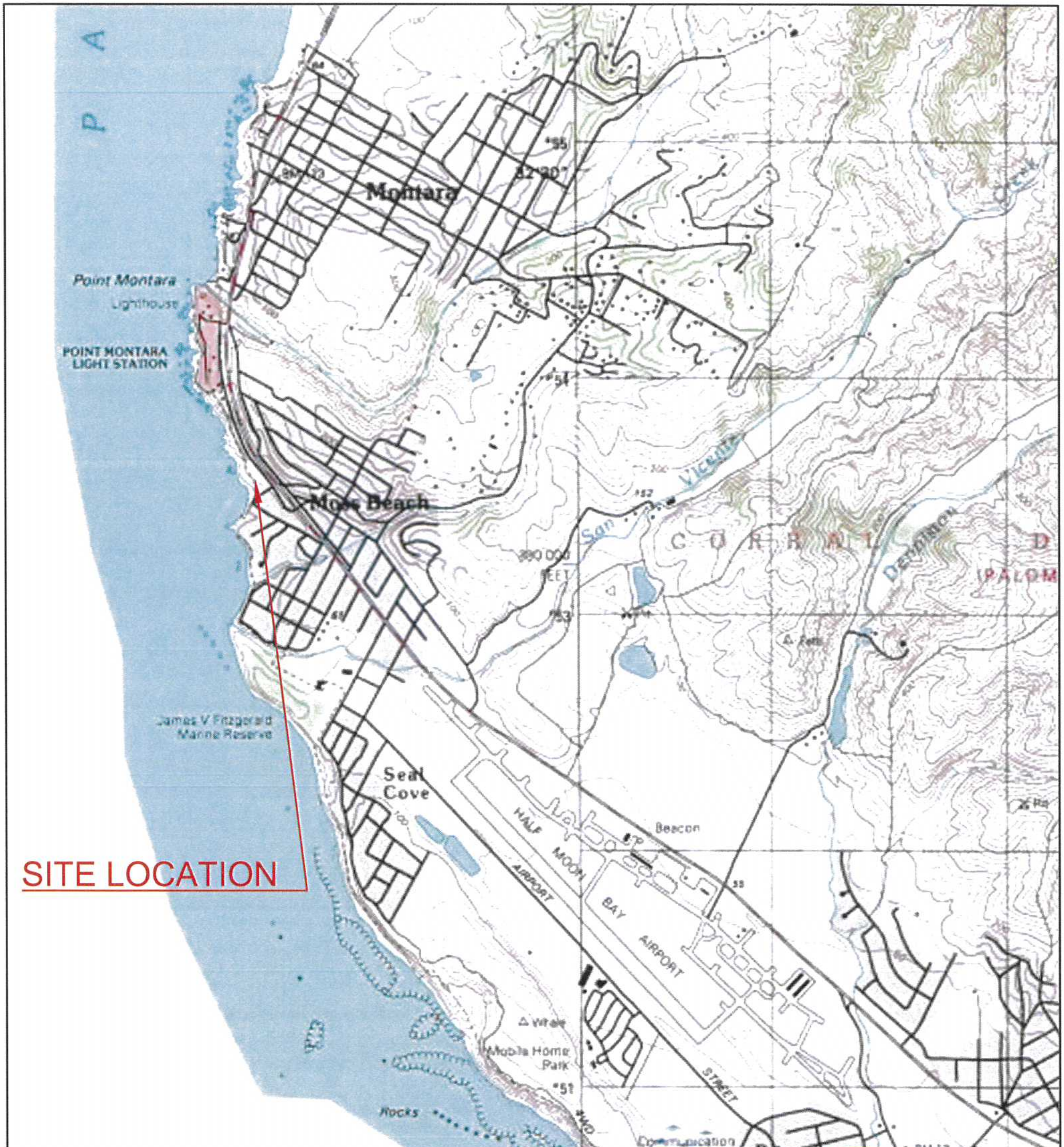
2. This report is issued with the understanding that it is the duty and responsibility of the owner or his representative or agent to ensure that the recommendations contained in this report are brought to the attention of the architects and engineers and contractors for the project, incorporated into the plans and specifications, and that the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

3. The conclusions and recommendations contained herein are professional opinions derived in accordance with current standards of professional practice in the Santa Cruz County area. No other warranty, expressed or implied, is made.

4. If any unexpected variations in soil conditions, or if adverse soil conditions are encountered during construction, or if the proposed construction will differ from that planned at the present time, Haro, Kasunich and Associates should be notified so that supplemental recommendations can be given.

5. 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 are 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.

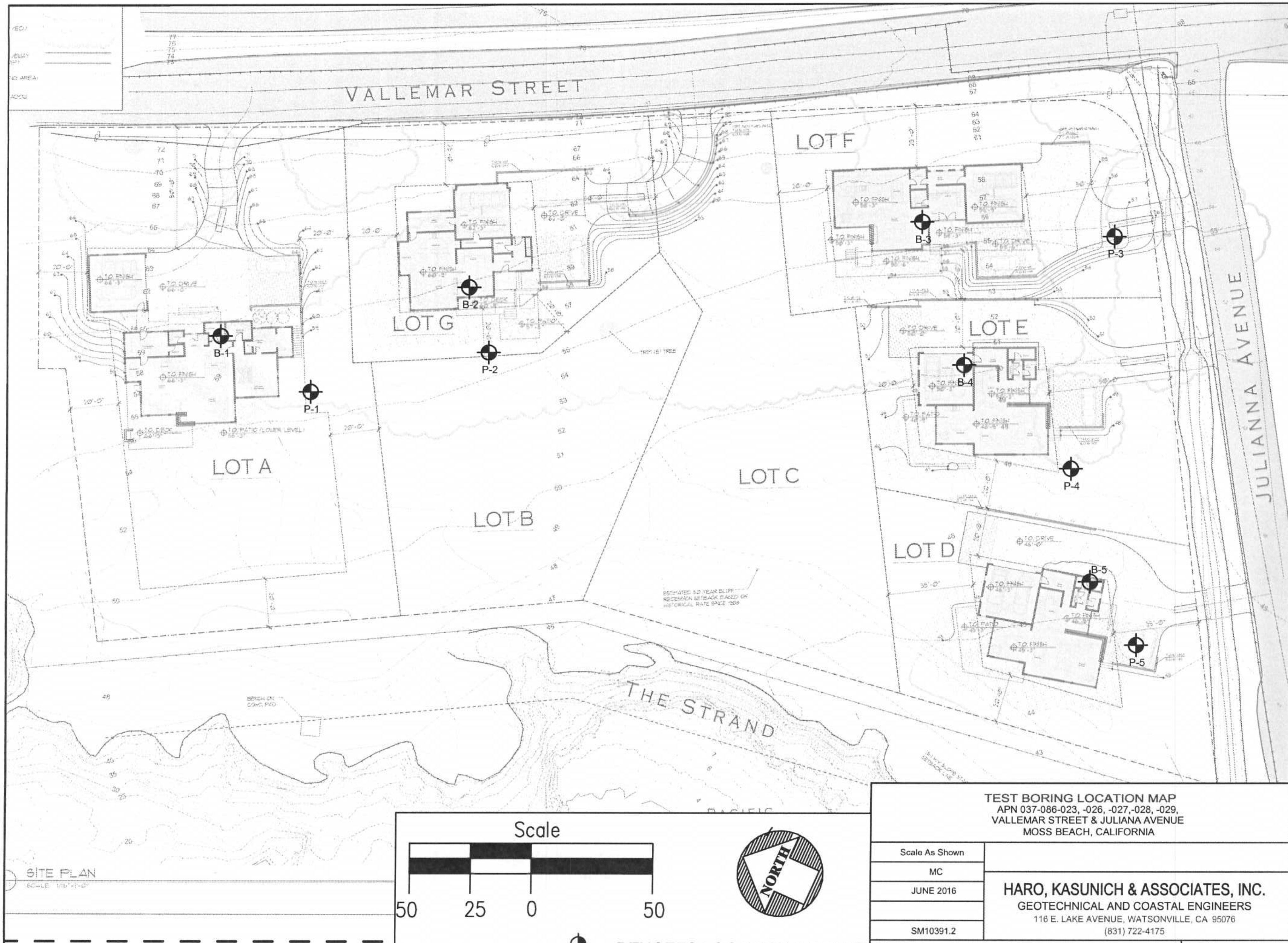
APPENDIX A



SITE VICINITY MAP
APN 037-086-023, -026, -027,-028, -029,
VALLEMAR STREET & JULIANA AVENUE
MOSS BEACH, CALIFORNIA

SCALE: No Scale	
DRAWN BY: MC	
DATE: JUNE 2016	**HARO, KASUNICH & ASSOCIATES, INC.** GEOTECHNICAL AND COASTAL ENGINEERS 116 E. LAKE AVENUE, WATSONVILLE, CA 95076 (831) 722-4175
REVISED:	
JOB NO: SM10391.2	

FIGURE NO. 1 | SHEET NO: 49



DENOTES LOCATION OF TEST BORING

TEST BORING LOCATION MAP
APN 037-086-023, -026, -027,-028, -029,
VALLEMAR STREET & JULIANA AVENUE
MOSS BEACH, CALIFORNIA

Scale As Shown	
MC	
JUNE 2016	HARO, KASUNICH & ASSOCIATES, INC. GEOTECHNICAL AND COASTAL ENGINEERS 116 E. LAKE AVENUE, WATSONVILLE, CA 95076 (831) 722-4175
SM10391.2	

FIGURE NO. 2

SHEET NO 50

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

GRAIN SIZES

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

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

RELATIVE DENSITY

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

CONSISTENCY

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

SAMPLING METHOD

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

H_2O

Final	▽
Initial	▼
Water level designation	

*Number of blows of 140 lb hammer falling 30 inches to drive a 2" O.D. (1⅜" I.D.) split spoon sampler (ASTM D-1586)
**Unconfined compressive strength in tons/ft² as determined by laboratory testing or approximated by the Standard Penetration Test (ASTM D-1586), pocket penetrometer, torvane, or visual observation.

Haro Kasunich & Associates	KEY TO LOGS VALLEMAR STREET & JULIANA AVENUE MOSS BEACH, CALIFORNIA	Project No. SM10391.2 JUNE 2016	Figure No. 3

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

Vallemar Street

PROJECT NO. SM10391.2

LOGGED BY MC **DATE DRILLED** 3-22-16 **BORING DIAMETER** 4" **BORING NO.** B-1

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0			Iceplant/roots	SM-CL					
	1-1-1 (L)		Brown Silty SAND with CLAY bindfer fine to medium grain, 1/4 inch clasts, wet, loose		19		113	12.7	(1-1) Atterberg Limits PI = 9 LL = 18.4%
	1-2 (T)		Brown orange Sandy CLAY, fine to medium grain, moist, very stiff	CL	21			17.2	
5	1-3-1 (L)		Same as above		36				(1-2) PI = 25 LL = 36.8
	1-4 (T)		Grading to a brown orange Clayey SAND, fine to medium grain, damp to moist, medium dense	SC	28				
10	1-5-1 (L)		Brown tan grey Silty SAND, fine to medium grain, damp to dry, cemented, very dense terrace deposit		50/6"				
	1-6 (T)		harder drilling at 11 feet		49				
			Brown Silty SAND with CLAY binder, fine to medium grain, damp, very dense						
15			Boring terminated at 13.5 feet						
20									
25									
30									
35									

SuperLog CivilTech Software, USA www.civiltech.com File: C:\superlog4\HKALOGS\SM10391.2 Vallemar Street.log Date: 6/27/2016

HARO, KASUNICH AND ASSOCIATES, INC.

BY: sr | FIGURE NO. 4

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

Vallemar Street

PROJECT NO. SM10391.2

LOGGED BY MC DATE DRILLED 3-22-16 BORING DIAMETER 4" BORING NO. B-2

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0			Topsoil/roots						
	2-1-1 (L)		Dark brown Clayey SAND, fine to medium grain, wet, loose	SC	20			15.3	Atterberg Limits PI = 22 LL = 31.5%
	2-2 (T)		Mottled brown orange grey Sandy CLAY, fine grain, moist, very stiff	CL	24				
5	2-3- (L)		Same as above		32				
	2-4 (T)		Same as above		26				
10	2-5-1 (L)		Grey with orange mottling Sandy CLAY, fine to medium grain, moist, very stiff coastal terrace	CL	38				
	2-6 (T)				22				
15	2-7 (T)		Grey with orange and rust mottling Silty SAND, fine to medium grain, damp, dense, terrace	SM	48				
20	2-8 (T)		Grey fine to medium SAND, wet to saturated, dense	SW	35				
			Boring terminated at 21.5 feet						
25									
30									
35									

HARO, KASUNICH AND ASSOCIATES, INC.

BY: sr FIGURE NO. 5

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

Vallemar Street

PROJECT NO. SM10391.2

LOGGED BY MC **DATE DRILLED** 3-22-16 **BORING DIAMETER** 4" **BORING NO.** B-3

Depth, ft.	Sample No. and type	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0		Roots/topsoil						
		Black Sandy CLAY, fine to medium grain, wet, firm	CL					
	3-1-1 (L)			19		110	17.2	Atterberg Limits PI = 18 LL = 30.3%
	3-2 (T)			19				
		Brown orange grey Sandy CLAY, fine to medium grain, moist, stiff, terrace deposit	CL					
5	3-3-1 (L)			35			15.5	PI = 28 LL = 37.9%
	3-4 (T)	Brown orange grey Clayey SAND, fine to coarse grain, moist to damp, medium dense	SC	28				
10	3-5- (L)	Same as above		53				
15								
		Grey fine SAND with SILT, saturated	SP-ML					
20	3-6 (T)			10				
		Boring terminated at 21.5 feet						
25								
30								
35								

SuperLog CivilTech Software, USA www.civiltech.com File: C:\superlog4\HKALOGS\SM10391.2 Vallemar Street.log Date: 6/27/2016

HARO, KASUNICH AND ASSOCIATES, INC.

BY: sr FIGURE NO. 6

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

Vallemar Street

PROJECT NO. SM10391.2

LOGGED BY MC — DATE DRILLED 3-24-16 — BORING DIAMETER 4" — BORING NO. B-4

Depth, ft.	Sample No. and type	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0		Dark brown Sandy CLAY, fine to medium grain, saturated, very loose	CL					
	4-1-1 (L)			6			18.8	Atterberg Limits Pi = 38 LL = 48.3%
		Brown CLAY, wet, firm	CL					
	4-2 (T)	Grey light brown sandy Lean CLAY, fine to medium grain, moist, very stiff, terrace	CL	12			15.5	
5	4-3-1 (L)	Same as above		34				(4-2) PI = 27 LL = 36.6%
	4-4 (T)	Same as above		22				(4-3) Unconfined Compression Test Qu = 8003 psf C = 2000 psf ϕ = 10°
		Grey Clayey SAND interbedded, moist with orange Clayey SILT, saturated	SC-ML					
10	4-5-1 (L)	Grey with light orange mottling Clayey SAND, fine to medium grain, moist, medium dense	SC	25				
15	4-6-1 (L)	Blue grey SAND with SILT, fine to medium grain, wet to saturated, weathered rock, medium dense to dense	SW-ML	53				
20								
		Harder drilling at 22 feet then broke through to soft gain						
25		Drill auger refusal at 25 feet Boring terminated at 25.01 feet						
30								
35								

HARO, KASUNICH AND ASSOCIATES, INC.

BY: sr — FIGURE NO. 7

SuperLog CivilTech Software, USA www.civiltech.com File: C:\superlog4\HKALOGS\SM10391.2 Vallemar Street.log Date: 6/27/2016

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

Vallemar Street

PROJECT NO. SM10391.2

LOGGED BY MC **DATE DRILLED** 3-24-16 **BORING DIAMETER** 4" **BORING NO.** B-5

Depth, ft.	Sample No. and type	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0		Iceplant and topsoil						
	5-1-1 (L)	Dark brown Clayey SAND, fine to medium grain, saturated, very loose	SC	7				
	5-2 (T)	Dark brown CLAY, moist, firm	CL	8				
		Brown Sandy CLAY, fine to medium grain, moist, medium dense, terrace	CL					
5	5-3-1 (L)	Grey orange mottling, Sandy CLAY, fine to coarse, grain, moist, very stiff	CL	36				
	5-4 (T)			20				
		Brown orange SAND, fine to medium grain, wet, medium dense	SW					
10	5-5-1 (L)			50		97	15.6	Saturated Direct Shear Test C = 0 psf $\phi = 43^{\circ}$
		Blue grey fine to medium SAND, saturated, dense	SW					
15	5-6 (T)			29				
		Boring terminated at 16.5 feet						
20								
25								
30								
35								

HARO, KASUNICH AND ASSOCIATES, INC.

BY: sr

FIGURE NO. 8

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

Vallemar Bluff

PROJECT NO. SM10391.2

LOGGED BY JD **DATE DRILLED** March 30, 2016 **BORING DIAMETER** 6" **BORING NO.** P-1

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0			Dark brown Silty SAND with Clay and Organics, roots, moist, loose (Topsoil)	TP					
			Dark brown Silty SAND, fine to medium grain, moist (Terrace)	SM					
2			Boring terminated at 2.1 feet						
4									
6									
8									
10									
12									
14									

SuperLog CivilTech Software, USA www.civiltech.com File: C:\Superlog4\HKALOGS\SM10391.2 Vallemar Bluff.log Date: 6/27/2016

HARO, KASUNICH AND ASSOCIATES, INC.

BY: dk

FIGURE NO. 9

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

Vallemar Bluff

PROJECT NO. SM10391.2

LOGGED BY	DATE DRILLED	BORING DIAMETER	BORING NO.
JD	March 30, 2016	6"	P-2

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0			Dark brown Silty SAND with Clay and Organics, roots, moist, loose (Topsoil)	TP					
			Dark brown Silty SAND, fine to medium grain, moist (Terrace)	SM					
2			Boring terminated at 2.1 feet						
4									
6									
8									
10									
12									
14									

HARO, KASUNICH AND ASSOCIATES, INC.

BY: dk

FIGURE NO. 10

Vallemar Bluff

PROJECT NO. SM10391.2

LOGGED BY JD DATE DRILLED March 30, 2016 BORING DIAMETER 6" BORING NO. P-3

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0			Dark brown Silty SAND, fine grain, roots, moist, loose (Topsoil)	TP					
			Dark brown Silty SAND, fine to medium grain, damp to moist, loose to medium dense	SM					
2									
4			Boring terminated at 4.0 feet						
6									
8									
10									
12									
14									

SuperLog CivilTech Software, USA www.civiltech.com File: C:\Superlog4\HKALOGS\SM10391.2 Vallemar Bluff.log Date: 6/27/2016

HARO, KASUNICH AND ASSOCIATES, INC.

BY: dk FIGURE NO. 11

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

Vallemar Bluff

PROJECT NO. SM10391.2

LOGGED BY JD | DATE DRILLED March 30, 2016 | BORING DIAMETER 6" | BORING NO. P-4

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0			Dark brown Silty SAND, fine grain, moist, roots, loose, topsoil	TP					
			Orange tan Sandy CLAY, moist, terrace	CL					
2		▼							
			Boring terminated at 2.9 feet						
4									
6									
8									
10									
12									
14									

SuperLog CivilTech Software, USA www.civiltech.com File: C:\Superlog4\HKALOGS\SM10391.2 Vallemar Bluff.log Date: 6/27/2016

HARO, KASUNICH AND ASSOCIATES, INC.

BY: dk | FIGURE NO. 12

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

Vallemar Bluff

PROJECT NO. SM10391.2

LOGGED BY JD | DATE DRILLED March 30, 2016 | BORING DIAMETER 6" | BORING NO. P-5

Depth, ft.	Sample No. and type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	Blows/foot 350 ft - lbs.	Qu - t.s.f. Penetrometer	Dry Density p.c.f.	Moisture % dry wt.	MISC. LAB RESULTS
0			Dark brown Silty SAND, fine grain, moist, roots, loose, topsoil	TP					
			Orange tan Sandy CLAY, moist, terrace	CL					
2									
4									
			Boring terminated at 4.5 feet						
6									
8									
10									
12									
14									

SuperLog CivilTech Software, USA www.civiltech.com File: C:\Superlog4\HKALOGS\SM10391.2 Vallemar Bluff.log Date: 6/27/2016

HARO, KASUNICH AND ASSOCIATES, INC.

BY: dk | FIGURE NO. 13

Direct Shear

Project:	Vallemar Bluff
Sample #	5-5-1
Description	Orange Brown Silty Sand

Date	4/7/2016
Tested By:	MA

Equation of Trendline	
Intercept	Slope
0	0.9312

Test Number	1	2	3	4
Normal Pressure (PSF)	530	1030	2030	4030
Max Shear Stress	13.4	33.5	64.7	
Shear Stress (PSF)	394.6	986.4	1902.2	

C (PSF)	PHI
0	43

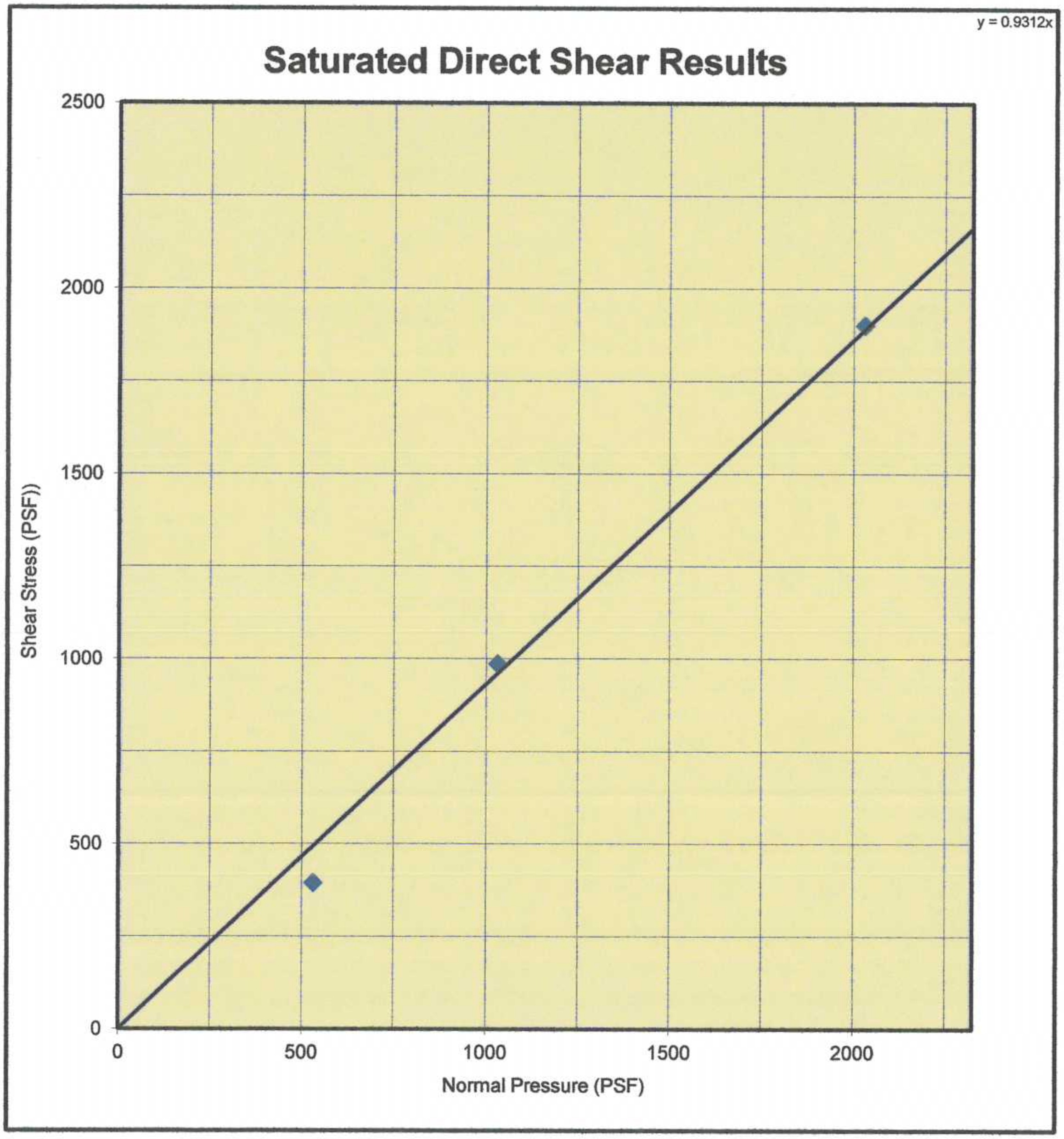
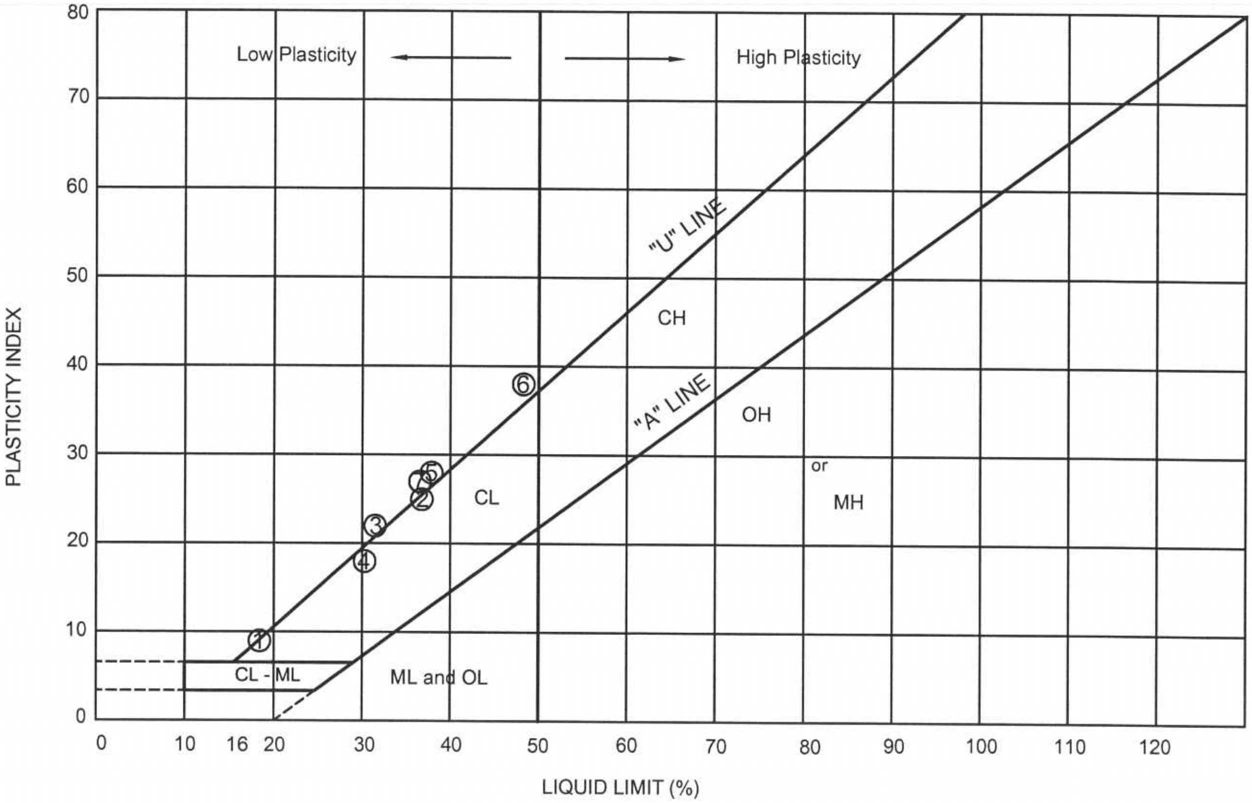


Figure No.	14

PLASTICITY CHART



PLASTICITY DATA

Key Symbol	Sample Number	Depth (feet)	Natural Water Content W(%)	Plastic Limit (%)	Liquid Limit (%)	Plasticity Index	Liquidity Index W - PL / LL - PL	Unified Soil Classification Symbol
①	1-1	2.0	12.7	10.3	18.4	9	+0.2667	CL
②	1-2	3.5	17.2	12.6	36.8	25	+0.1840	CL
③	2-1	2.0	15.3	9.9	31.5	22	+0.2455	CL
④	3-1	2.0	17.2	17.1	30.3	18	+0.0056	CL
⑤	3-3	5.0	15.5	10.2	37.9	28	+0.1893	CL
⑥	4-1	2.5	18.8	10.8	48.3	38	+0.2105	CL
⑦	4-2	3.5	15.5	9.7	36.6	27	+0.2148	CL

ATTERBERG LIMITS TEST RESULTS
APN 037-086-023, -026, -027,-028, -029,
VALLEMAR STREET & JULIANA AVENUE
MOSS BEACH, CALIFORNIA

SCALE: No Scale
DRAWN BY: MC
DATE: JUNE 2016
REVISED:
JOB NO: SM10391.2

HARO, KASUNICH & ASSOCIATES, INC.
GEOTECHNICAL AND COASTAL ENGINEERS
116 E. LAKE AVENUE, WATSONVILLE, CA 95076
(831) 722-1475

FIGURE NO. 15

SHEET NO. 63

ANALYTICAL CHEMISTS
and
BACTERIOLOGISTS
Approved by State of California

SOIL CONTROL LAB

42 HANGAR WAY
WATSONVILLE
CALIFORNIA
95076
USA

TEL: 831-724-5422
FAX: 831-724-3188

Work Order #: 6040263
Account #: 2953
Date Received: April 7, 2016
Date Reported: April 11, 2016

Haro - Kasunich and Assoc.
116 East Lake Avenue
Watsonville, CA 95076

Reporting Date: April 11, 2016

Date Received: April 7, 2016
Project#/Name: SM 10391.2 / Vallemar Bluff
Matrix: Soil

Sample Identification	pH (units)	Chloride (mg/Kg)	Sulfate (mg/Kg)	Resistivity (ohms x cm)
LOT A	4.4	470	130	240
LOT E	5.8	540	290	240
	Method CalTest 643 June 2007	**Method** CalTest 422 April 2000	**Method** CalTest 417 March 1999	**Method** CalTest 643 June 2007

Mike Galloway

Figure No. 16

APPENDIX B

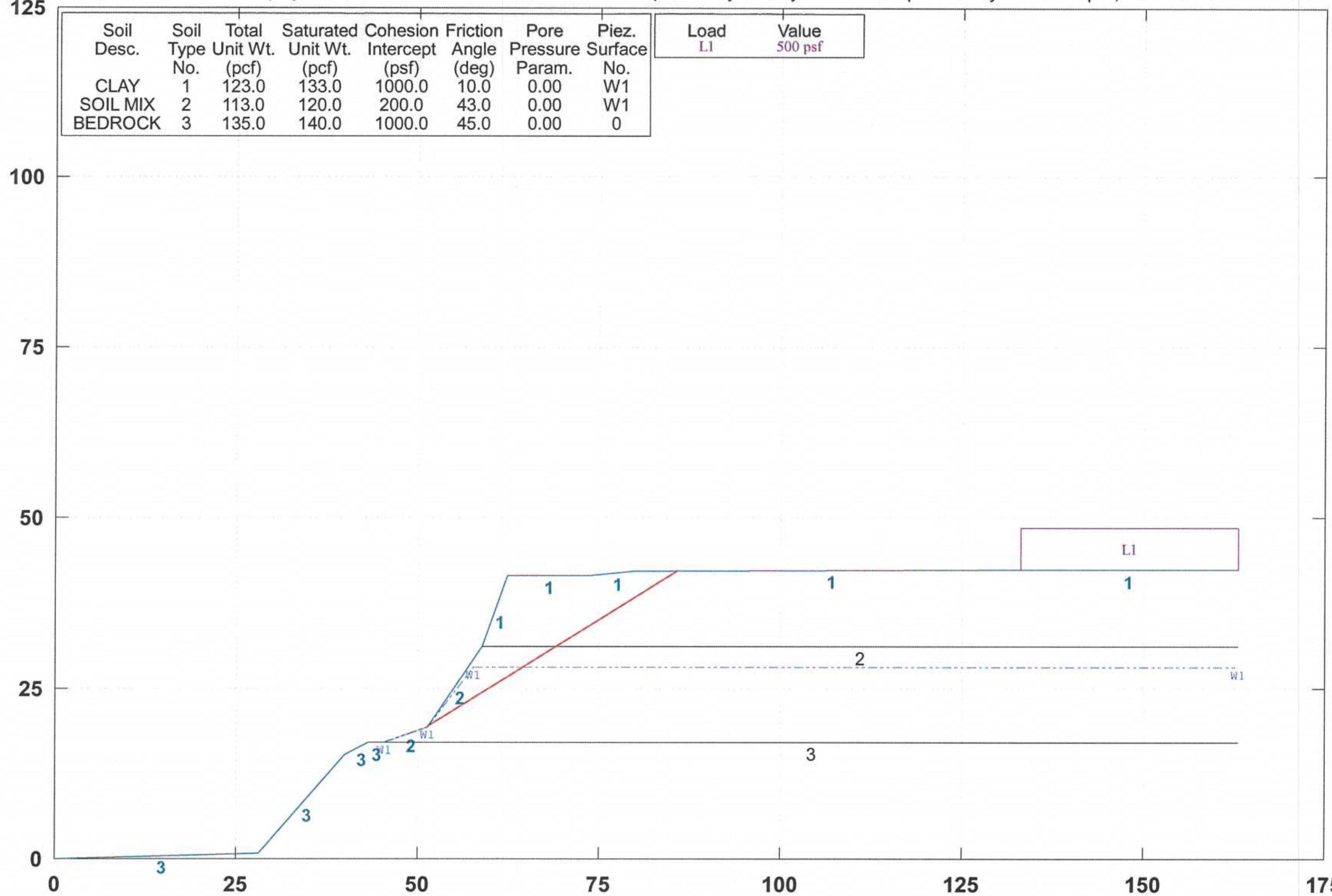
Slope Stability Analysis Results

VALLEMAR ST AND JULIANA AVE SLOPE ST ABILITY ANALYSIS SECTION 3 STATIC

c:\users\moses\documents\projects\san mateo\coastal bluff\vallemar bluff\slope stability\stability section 3 stc.plt Run By: Moses Cuprill, P.E. 6/25/2016 06:14PM

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Piez. Surface No.
CLAY	1	123.0	133.0	1000.0	10.0	0.00	W1
SOIL MIX	2	113.0	120.0	200.0	43.0	0.00	W1
BEDROCK	3	135.0	140.0	1000.0	45.0	0.00	0

Load	Value
L1	500 psf



GSTABL7 v.2 FSmin=2.48

Factor Of Safety Is Calculated By The Simplified Janbu Method

66

***** GSTABL7 *****

** GSTABL7 by Dr. Garry H. Gregory, Ph.D.,P.E.,D.GE **
** Original Version 1.0, January 1996; Current Ver. 2.005.3, Feb. 2013 **

****
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: 6/25/2016
Time of Run: 06:14PM
Run By: Moses Cuprill, P.E.
Input Data Filename: C:\Users\Moses\Documents\Projects\San Mateo\Coastal Bluff\Vallemar Bluff\sLOPE sTABILITY\stability section 3 stc.in
Output Filename: C:\Users\Moses\Documents\Projects\San Mateo\Coastal Bluff\Vallemar Bluff\sLOPE sTABILITY\stability section 3 stc.OUT
Unit System: English
Plotted Output Filename: C:\Users\Moses\Documents\Projects\San Mateo\Coastal Bluff\Vallemar Bluff\sLOPE sTABILITY\stability section 3 stc.PLT
PROBLEM DESCRIPTION: VALLEMAR ST AND JULIANA AVE SLOPE ST
ABILITY ANALYSIS SECTION 3 STATIC

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	200.0	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 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 = 2.481

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	193.22	0.66
2	33.62	54.56	7.30	407.38	186.67
3	33.62	58.20	1.44	619.31	411.99
4	33.62	60.55	4.20	1203.90	719.26
5	33.62	63.47	2.82	1671.16	940.88
6	33.62	66.90	5.42	1479.07	795.25
7	33.62	71.43	5.45	1365.25	598.58
8	33.62	76.75	7.33	1285.98	381.50
9	33.62	82.76	7.11	1195.46	133.63

Sum of the Resisting Forces (including Pier/Pile, Tieback, Reinforcing
Soil Nail, and Applied Forces if applicable) = 47011.89 (lbs)
Average Available Shear Strength (including Tieback, Pier/Pile, Reinforcing,
Soil Nail, and Applied Forces if applicable) = 1144.05(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)

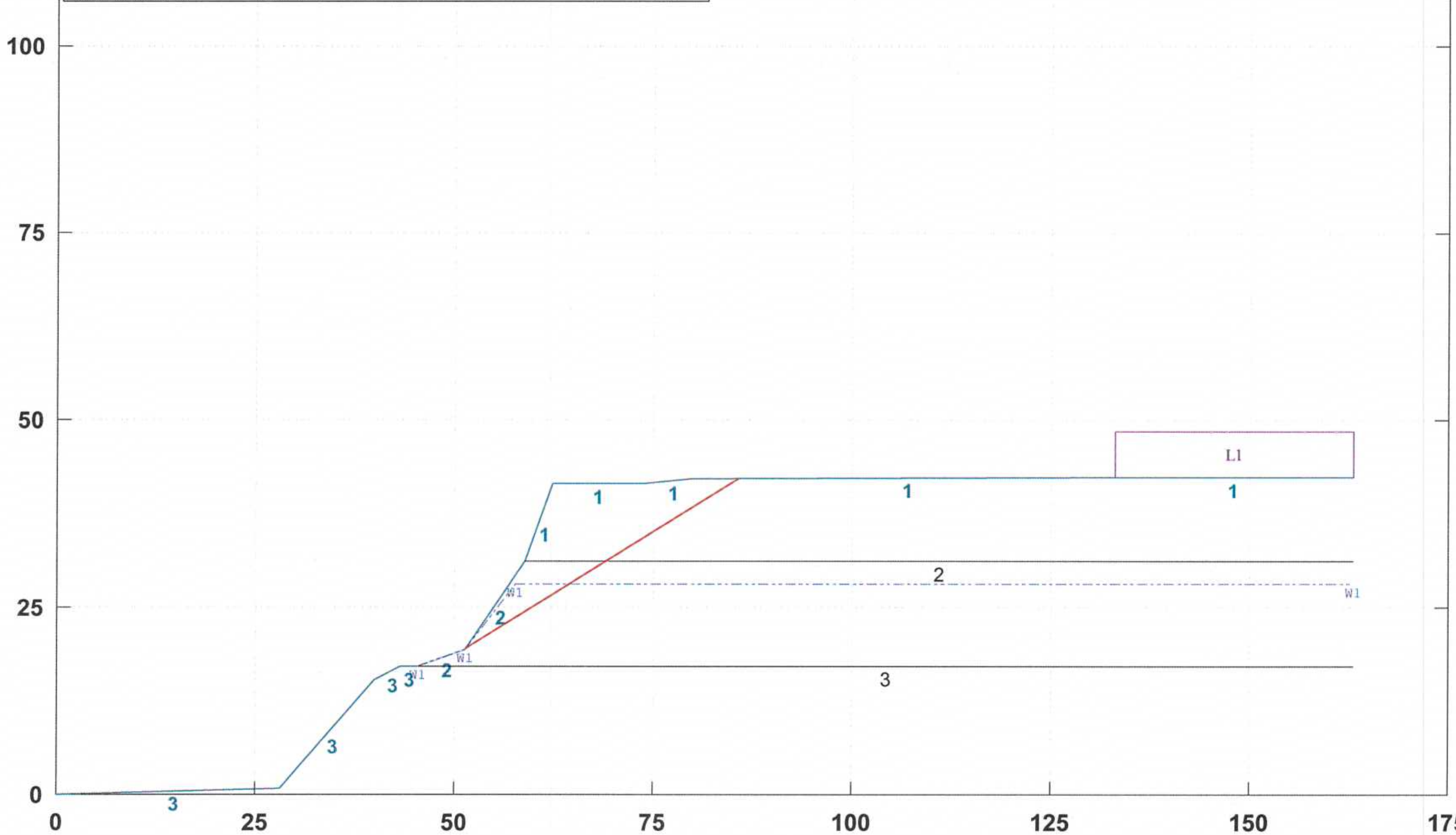
**** END OF GSTABL7 OUTPUT ****

VALLEMAR ST AND JULIANA AVE SLOPE ST ABILITY ANALYSIS SECTION 3 SEISMIC

c:\users\moses\documents\projects\san mateo\coastal bluff\vallemar bluff\slope stability\stability section 3 smc.plt Run By: Moses Cuprill, P.E. 6/25/2016 06:13PM

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Piez. Surface No.
CLAY	1	123.0	133.0	1000.0	10.0	0.00	W1
SOIL MIX	2	113.0	120.0	200.0	43.0	0.00	W1
BEDROCK	3	135.0	140.0	1000.0	45.0	0.00	0

Load	Value
L1	500 psf
Peak(A)	0.890(g)
kh Coef.	0.510(g)<



GSTABL7 v.2 FSmin=1.27
Factor Of Safety Is Calculated By The Simplified Janbu Method

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

***** GSTABL7 *****

** GSTABL7 by Dr. Garry H. Gregory, Ph.D.,P.E.,D.GE **
** Original Version 1.0, January 1996; Current Ver. 2.005.3, Feb. 2013 **

****

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: 6/25/2016
Time of Run: 06:13PM
Run By: Moses Cuprill, P.E.
Input Data Filename: C:\Users\Moses\Documents\Projects\San Mateo\Coastal Bluff\Vallemar Bluff\sLOPE sTABILITY\stability section 3 smc.in
Output Filename: C:\Users\Moses\Documents\Projects\San Mateo\Coastal Bluff\Vallemar Bluff\sLOPE sTABILITY\stability section 3 smc.OUT
Unit System: English
Plotted Output Filename: C:\Users\Moses\Documents\Projects\San Mateo\Coastal Bluff\Vallemar Bluff\sLOPE sTABILITY\stability section 3 smc.PLT
PROBLEM DESCRIPTION: VALLEMAR ST AND JULIANA AVE SLOPE ST
ABILITY ANALYSIS SECTION 3 SEISMIC

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	200.0	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
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 = 1.270

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	1045.1	0.0	0.0
3	1.2	893.0	0.0	385.3	0.0	0.0	455.4	0.0	0.0
4	3.5	4547.0	0.0	714.0	0.0	0.0	2319.0	0.0	0.0
5	2.3	3985.3	0.0	137.0	0.0	0.0	2032.5	0.0	0.0
6	4.5	6481.7	0.0	0.0	0.0	0.0	3305.7	0.0	0.0
7	4.5	4911.2	0.0	0.0	0.0	0.0	2504.7	0.0	0.0
8	6.1	4203.4	0.0	0.0	0.0	0.0	2143.7	0.0	0.0
9	5.9	1428.9	0.0	0.0	0.0	0.0	728.7	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	162.26	1.17
2	33.62	54.56	7.30	342.11	329.87
3	33.62	58.20	1.44	520.08	728.04
4	33.62	60.55	4.20	1011.01	1271.02
5	33.62	63.47	2.82	1403.40	1662.64
6	33.62	66.90	5.42	1242.09	1405.29
7	33.62	71.43	5.45	1308.90	1057.76
8	33.62	76.75	7.33	1232.89	674.16
9	33.62	82.76	7.11	1146.11	236.14

Sum of the Resisting Forces (including Pier/Pile, Tieback, Reinforcing Soil Nail, and Applied Forces if applicable) = 42507.43 (lbs)
Average Available Shear Strength (including Tieback, Pier/Pile, Reinforcing, Soil Nail, and Applied Forces if applicable) = 1034.43(psf)
Sum of the Driving Forces = 33482.20 (lbs)
Average Mobilized Shear Stress = 814.80(psf)
Total length of the failure surface = 41.09(ft)

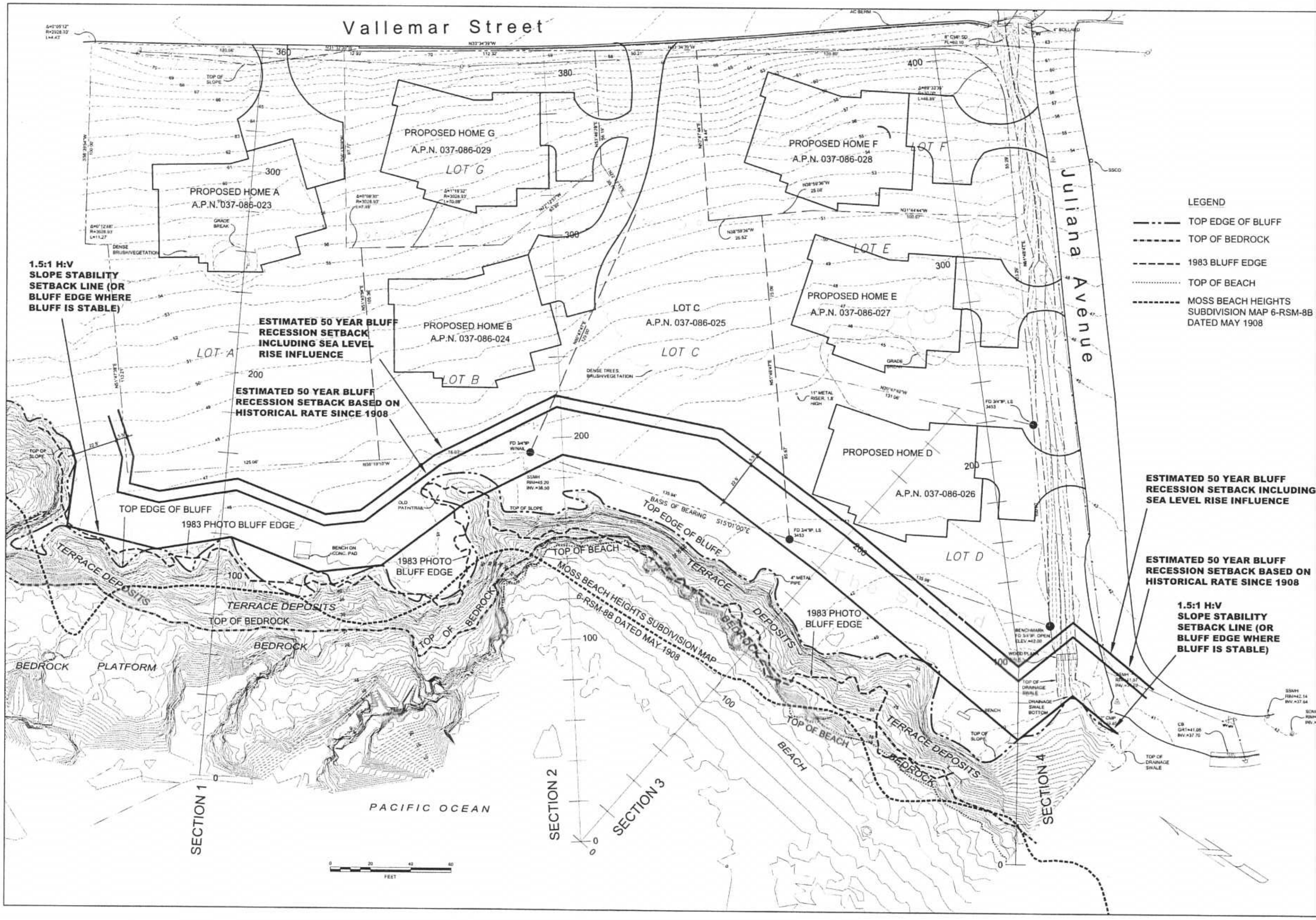
*** SEISMIC SLOPE DISPLACEMENT DATA ***

(Note: kv is set = zero for displacement calculations)
Seismic Yield Coefficient (ky) = 0.7575(g)
Calculated Newmark Seismic Displacement = 0.119(ft)
Non-Symmetrical Sliding Resistance Has Been Specified for Downhill Sliding.

**** END OF GSTABL7 OUTPUT ****

APPENDIX C

Coastal Bluff Recession Map and Sections



COASTAL BLUFF RECESSION MAP
MOSS BEACH ASSOCIATES, LLC
VALLEMAR STREET & JULIANA AVENUE, MOSS BEACH, CA
SAN MATEO COUNTY A.P.N.'S 037-086-023, 024, 025, 026, 027, 028 & 029

HARO, KASUNICH AND ASSOCIATES, INC.
CONSULTING CIVIL, GEOTECHNICAL & COASTAL ENGINEERS
116 EAST LAKE AVE., WATSONVILLE, CA 95076 (831) 722-4175

REVISIONS	BY
Date	3/9/2015
Scale	AS SHOWN
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Job	
Sheet	1 OF 2 SHEETS

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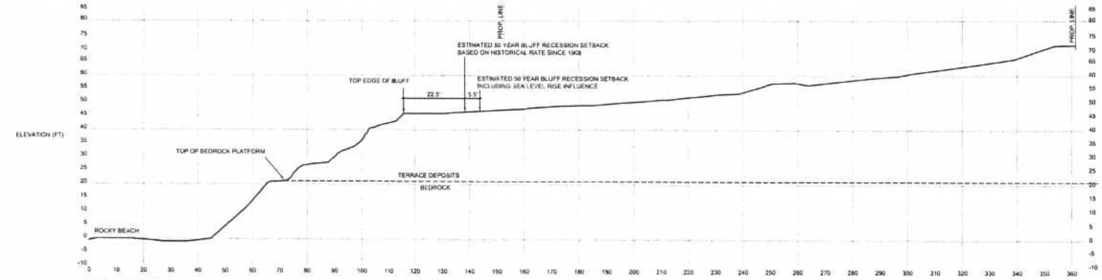

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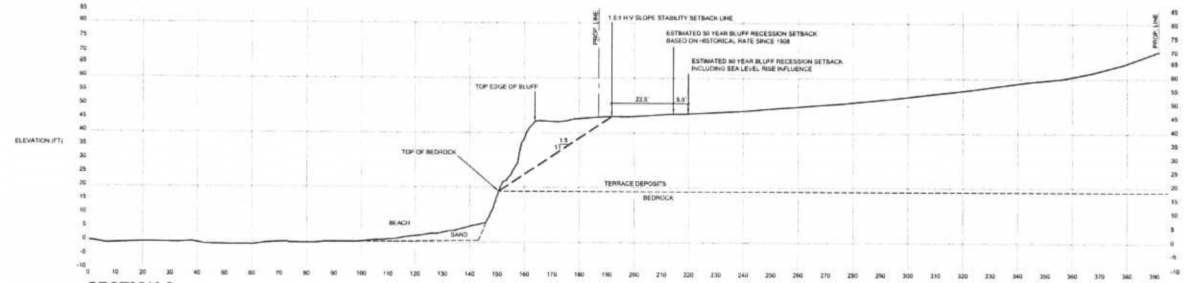

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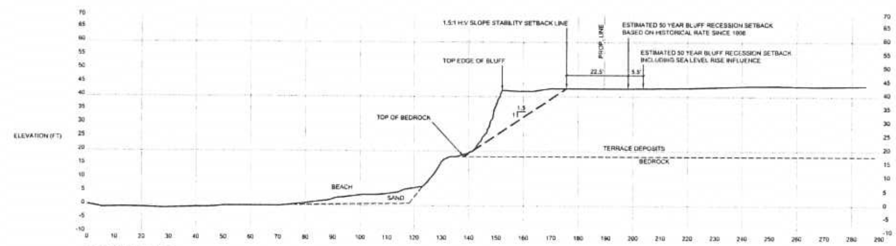

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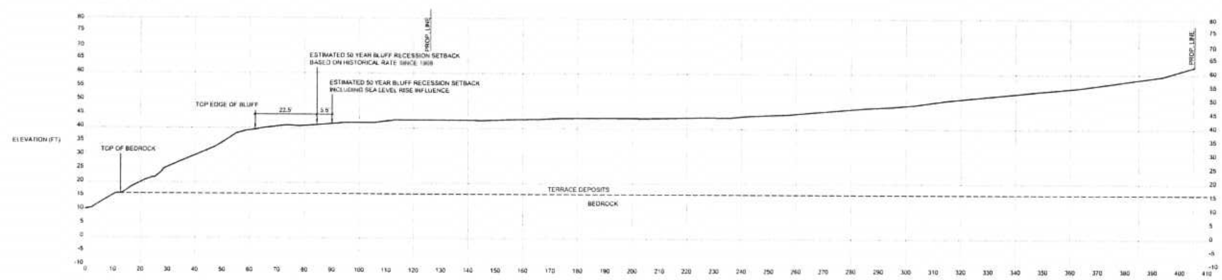

SECTION 4

REVISIONS	BY

COASTAL BLUFF RECESSION CROSS SECTIONS
MOSS BEACH ASSOCIATES, LLC
VALLEMAR STREET & JULIANA AVENUE, MOSS BEACH, CA
SAN MATEO COUNTY A.P.N.'S 037-086-023, 024, 025, 026, 027, 028 & 029

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Date	3/9/2015
Scale	AS SHOWN
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Sheet	2 OF 2 SHEETS

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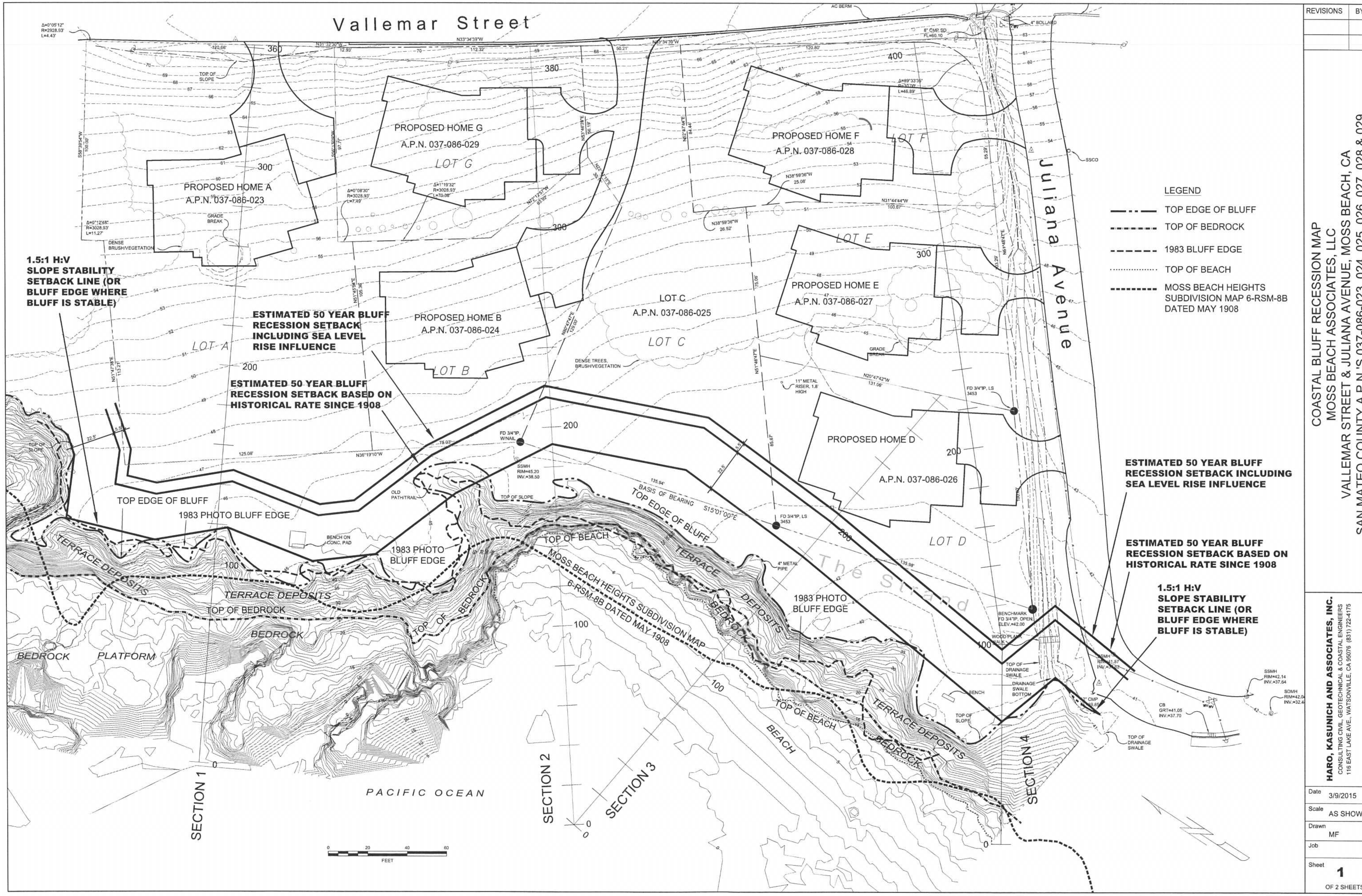

COASTAL BLUFF RECESSION MAP
MOSS BEACH ASSOCIATES, LLC
VALLEMAR STREET & JULIANA AVENUE, MOSS BEACH, CA
SAN MATEO COUNTY A.P.N.'S 037-086-023, 024, 025, 026, 027, 028 & 029
HARO, KASUNICH AND ASSOCIATES, INC.
CONSULTING CIVIL, GEOTECHNICAL & COASTAL ENGINEERS
116 EAST LAKE AVE., WATSONVILLE, CA 95076 (831) 722-4175
REVISIONS
BY
Date 3/9/2015
Scale AS SHOWN
Drawn MF
Job
Sheet 1
OF 2 SHEETS
LEGEND
TOP EDGE OF BLUFF
TOP OF BEDROCK
1983 BLUFF EDGE
TOP OF BEACH
MOSS BEACH HEIGHTS SUBDIVISION MAP 6-RSM-8B DATED MAY 1908
Vallemar Street
Juliana Avenue
The Strand
PROPOSED HOME A
A.P.N. 037-086-023
PROPOSED HOME B
A.P.N. 037-086-024
LOT C
A.P.N. 037-086-025
PROPOSED HOME D
A.P.N. 037-086-026
PROPOSED HOME E
A.P.N. 037-086-027
PROPOSED HOME F
A.P.N. 037-086-028
PROPOSED HOME G
A.P.N. 037-086-029
LOT A
LOT B
LOT C
LOT D
LOT E
LOT F
LOT G
1.5:1 H:V SLOPE STABILITY SETBACK LINE (OR BLUFF EDGE WHERE BLUFF IS STABLE)
ESTIMATED 50 YEAR BLUFF RECESSION SETBACK INCLUDING SEA LEVEL RISE INFLUENCE
ESTIMATED 50 YEAR BLUFF RECESSION SETBACK BASED ON HISTORICAL RATE SINCE 1908
TOP EDGE OF BLUFF
1983 PHOTO BLUFF EDGE
TERRACE DEPOSITS
TOP OF BEDROCK
BEDROCK
BEDROCK PLATFORM
TOP OF BEACH
BEACH
MOSS BEACH HEIGHTS SUBDIVISION MAP 6-RSM-8B DATED MAY 1908
BASIS OF BEARING S15°01'00"E
DENSE TREES, BRUSH/VEGETATION
DENSE BRUSH/VEGETATION
GRADE BREAK
BENCH ON CONC. PAD
OLD PATH/TRAIL
WOOD PLANK WALK
TOP OF DRAINAGE SWALE
DRAINAGE SWALE BOTTOM
BENCHMARK FD 3/4"IP, OPEN ELEV.=42.00
FD 3/4"IP, LS 3453
11" METAL RISER, 1.8' HIGH
4" METAL PIPE
SSMH RIM=45.20 INV.=38.50
PACIFIC OCEAN
SECTION 1
SECTION 2
SECTION 3
SECTION 4
FEET
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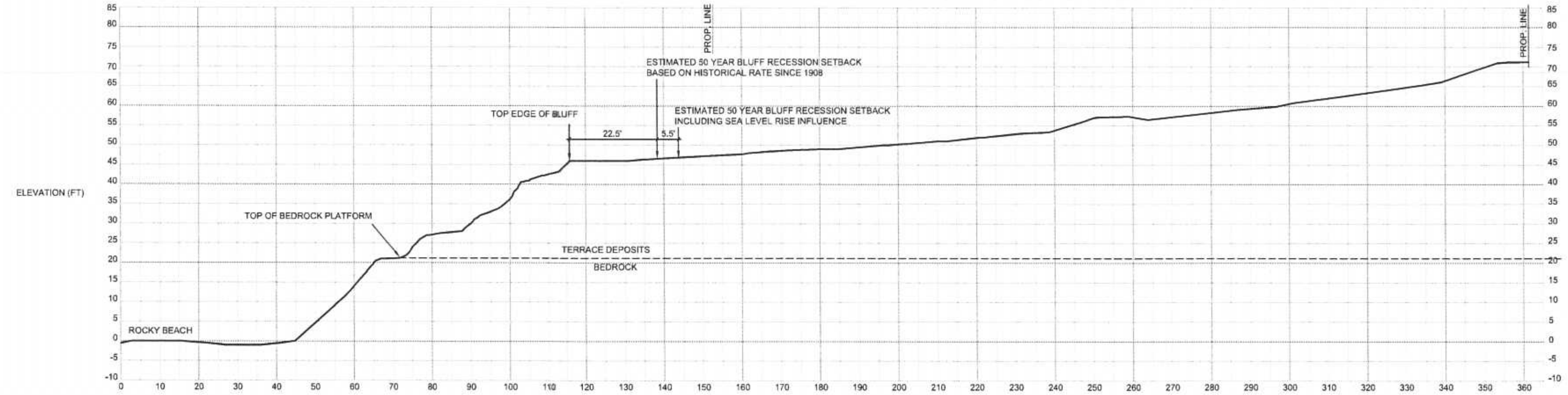
REVISIONS	BY



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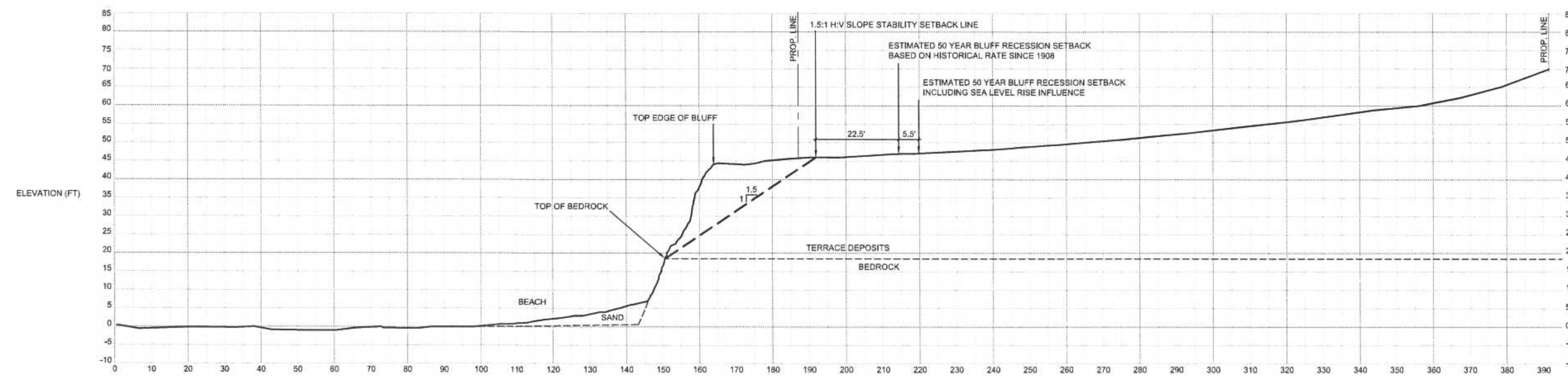

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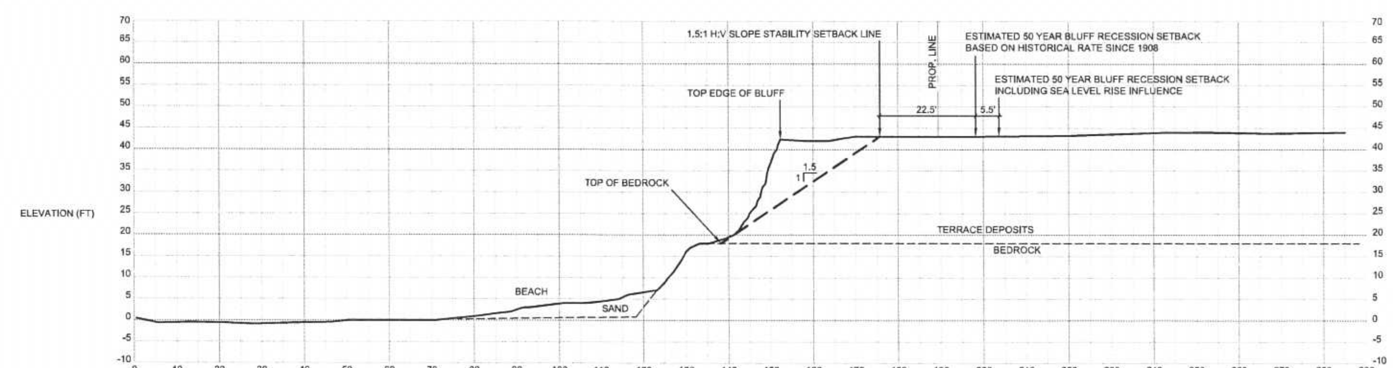

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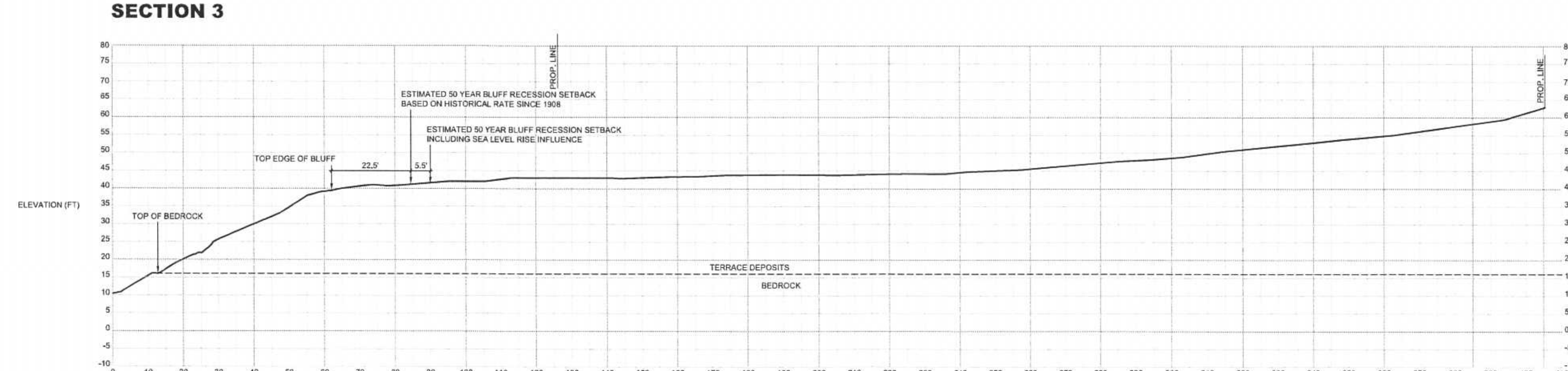

SECTION 4

COASTAL BLUFF RECESSION CROSS SECTIONS
MOSS BEACH ASSOCIATES, LLC
VALLEMAR STREET & JULIANA AVENUE, MOSS BEACH, CA
SAN MATEO COUNTY A.P.N.'S 037-086-023, 024, 025, 026, 027, 028 & 029

HARO, KASUNICH AND ASSOCIATES, INC.
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Date 3/9/2015
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Sheet 2 OF 2 SHEETS

76

APPENDIX D

Percolation Test Results

Percolation Test Results For Vallemar Street and Jullaina APN 037-086-023

Project No: SM10391.2
Date: 31 APRIL 2016
By: Haro, Kasunich and Associates

HOLE NO.:	P-1		TEST DATE:3/31/16 DRILL DATE: 3/30/16			
WATER LEVEL AFTER PRE-SOAK: Dry			DEPTH OF BORING (feet) 2.146			
TESTED BY:	JD		PERCOLATION ZONE (feet):		1.146	2.146
READING	ELAPSED TIME (min)	WATER DEPTH (feet)	REFILL TO (feet)	Incremental Change (in.)	PERCOLATION RATE (min/inch)	PERC (in/hr)
Start	0	1.6250	-	-	-	-
1	40	1.7917	1.6302	2.0000	20.00	3.0000
2	80	1.7708	1.6250	1.6875	23.70	2.5313
3 .	115	1.7917	1.6250	2.0000	17.50	3.4286
4	145	1.7813	1.6250	1.8750	16.00	3.7500
5	175	1.7760	1.6250	1.8125	16.55	3.6250
6	205	1.7813	1.6250	1.8750	16.00	3.7500
7	235	1.7865	1.6250	1.9375	15.48	3.8750
8	265	1.7917	1.6250	2.0000	15.00	4.0000

Average Of Reading's (in/hr)= 3.4950

Reported Percolation Rate (in/hr) = 3.9

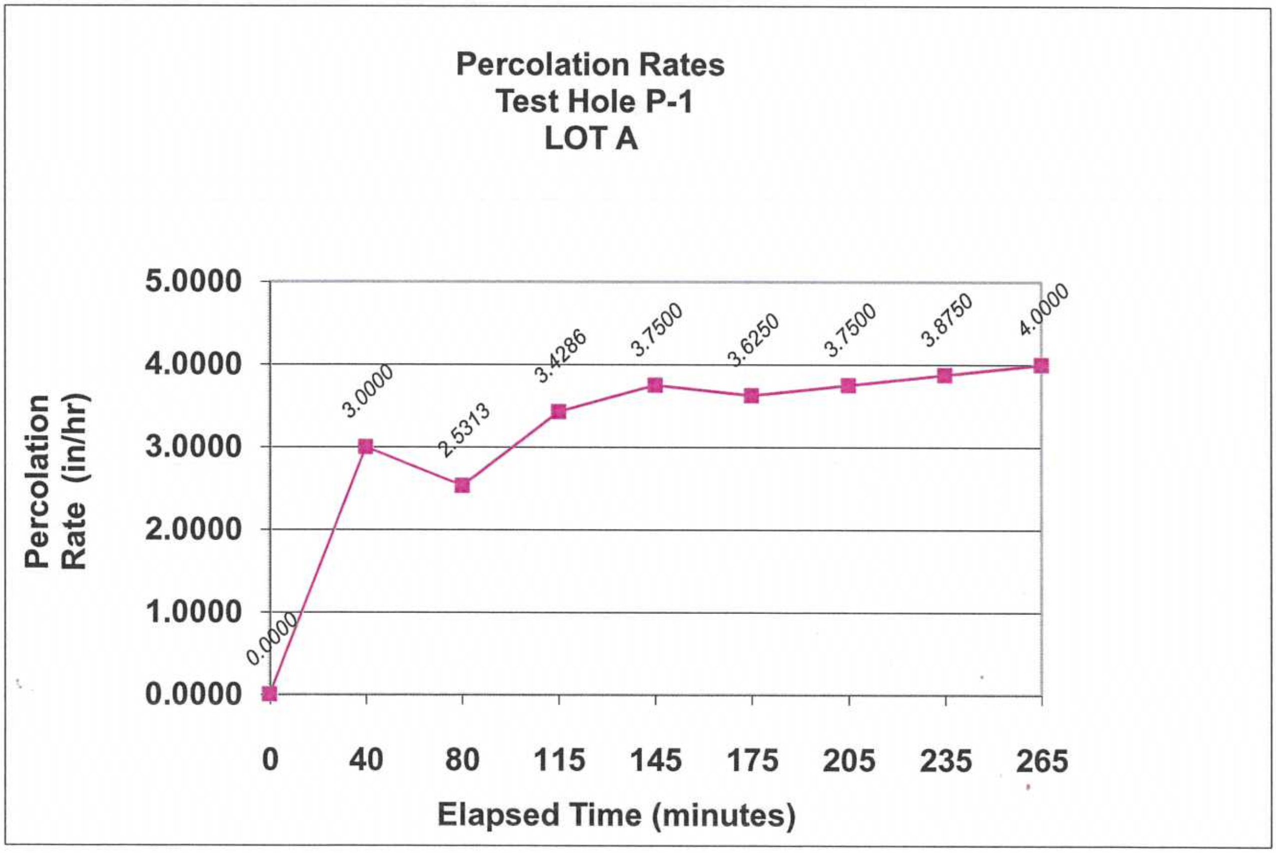


Figure No. ______

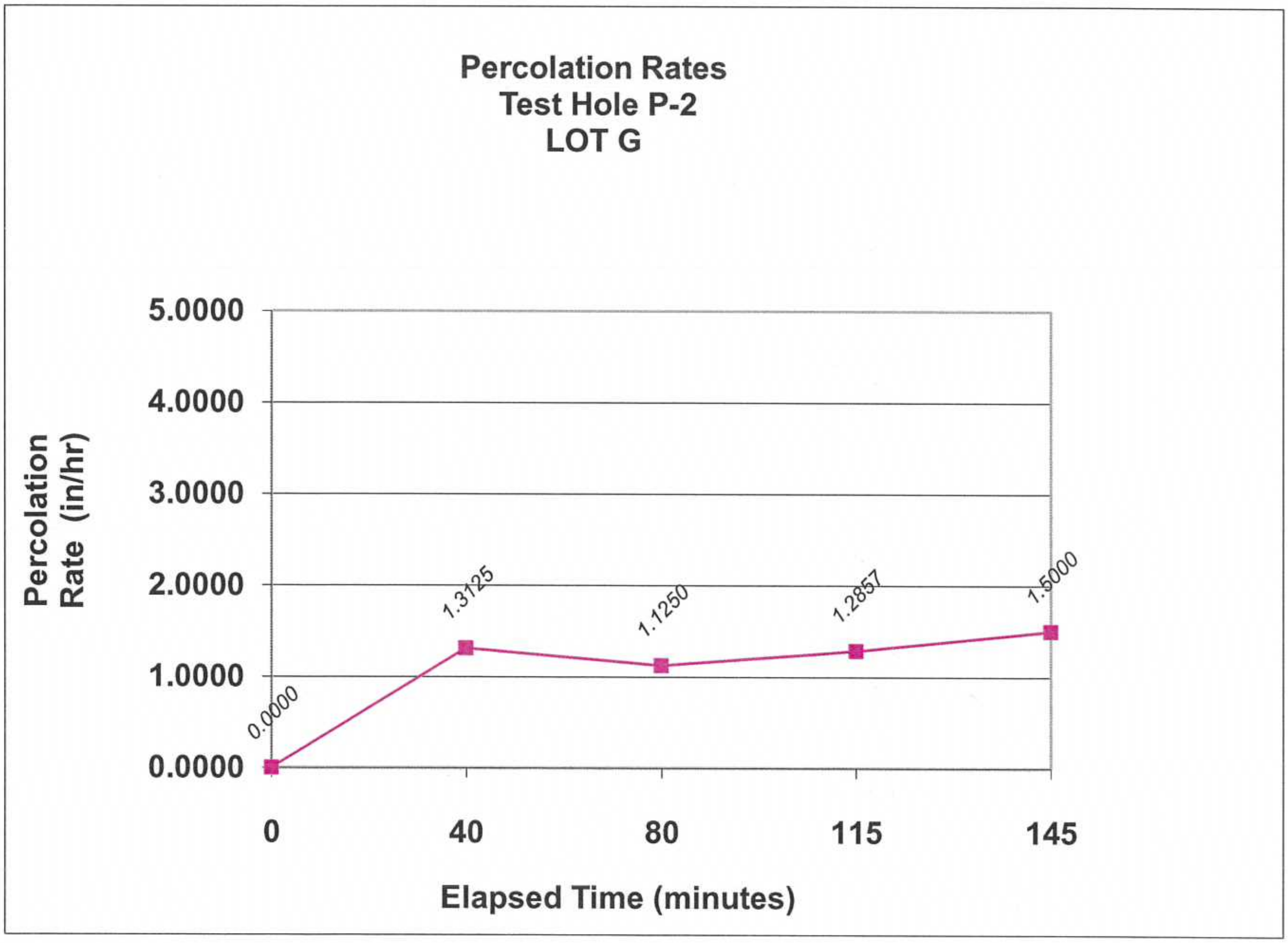
Percolation Test Results For Vallemar Street and Jullaina APN 037-086-029

Project No: SM10391.2
Date: 31 APRIL 2016
By: Haro, Kasunich and Associates

HOLE NO.:	P-2		TEST DATE:3/31/16 DRILL DATE: 3/30/16			
WATER LEVEL AFTER PRE-SOAK: Dry			DEPTH OF BORING (feet) 2.146			
TESTED BY:	JD		PERCOLATION ZONE (feet):		1.146	2.146
READING	ELAPSED TIME (min)	WATER DEPTH (feet)	REFILL TO (feet)	Incremental Change (in.)	PERCOLATION RATE (min/inch)	PERC (in/hr)
Start	0	1.6458	-	-	-	-
1	40	1.7188	1.6458	0.8750	45.71	1.3125
2	80	1.7083	1.6458	0.7500	53.33	1.1250
3	115	1.7083	1.6458	0.7500	46.67	1.2857
4	145	1.7083	1.6458	0.7500	40.00	1.5000
5	-	-	-	-	-	-
6	-	-	-	-	-	-
7	-	-	-	-	-	-
8	-	-	-	-	-	-

Average Of Reading's (in/hr)= 1.3058

Reported Percolation Rate (in/hr) = 1.4



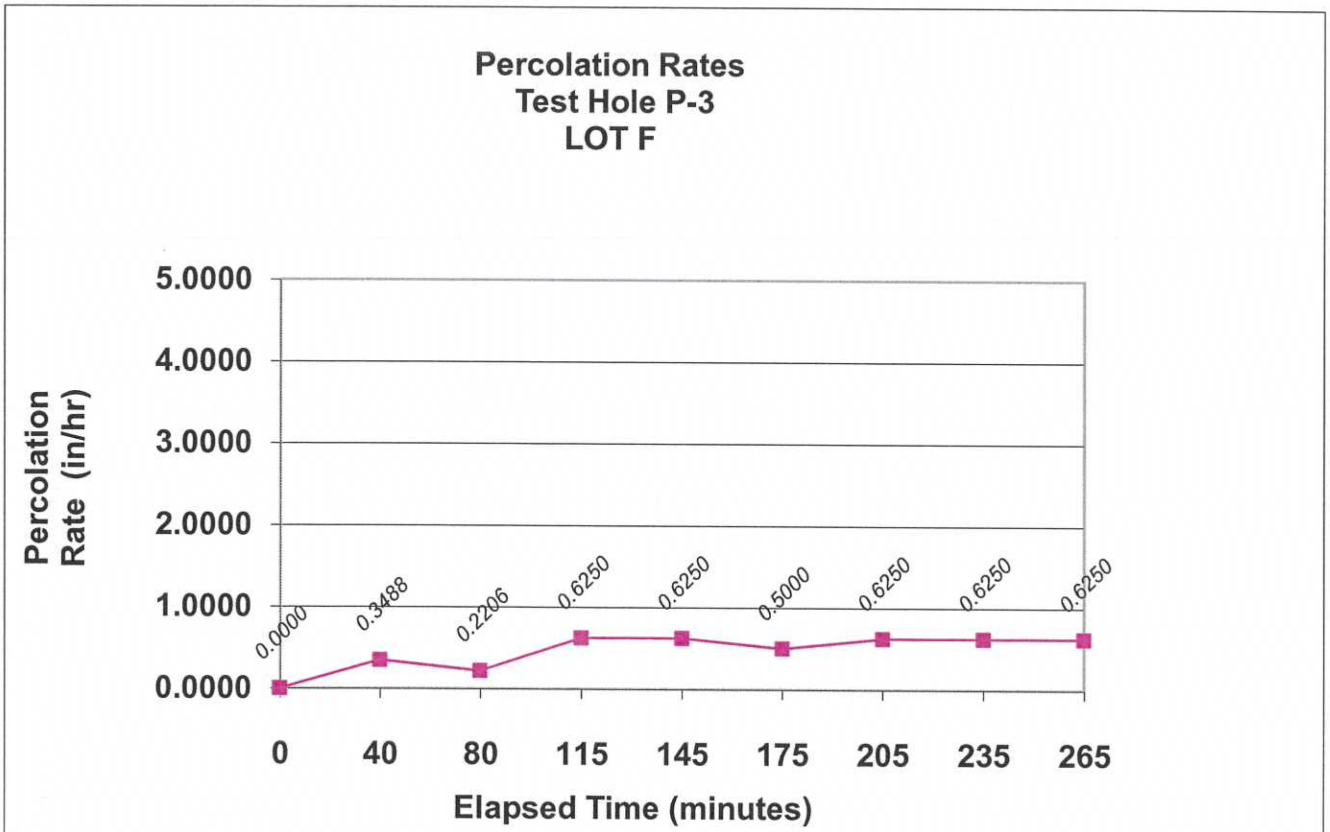
Percolation Test Results For Vallemar Street and Jullaina APN 037-086-028

Project No: SM10391.2
Date: 31 APRIL 2016
By: Haro, Kasunich and Associates

HOLE NO.:	P-3		TEST DATE:3/31/16 DRILL DATE: 3/30/16			
WATER LEVEL AFTER PRE-SOAK: 3.708			DEPTH OF BORING (feet) 4.042			
TESTED BY: JD			PERCOLATION ZONE (feet):		3.042	4.042
READING	ELAPSED TIME (min)	WATER DEPTH (feet)	REFILL TO (feet)	Incremental Change (in.)	PERCOLATION RATE (min/inch)	PERC (in/hr)
Start	0	3.5417	-	-	-	-
1	43	3.5625	3.5417	0.2500	172.00	0.3488
2	77	3.5521	3.5417	0.1250	272.00	0.2206
3	107	3.5677	3.5417	0.3125	96.00	0.6250
4	137	3.5677	3.5417	0.3125	96.00	0.6250
5	167	3.5625	3.5417	0.2500	120.00	0.5000
6	197	3.5677	3.5417	0.3125	96.00	0.6250
7	227	3.5677	3.5417	0.3125	96.00	0.6250
8	257	3.5677	3.5417	0.3125	96.00	0.6250

Average Of Reading's (in/hr)= 0.5243

Reported Percolation Rate (in/hr) = 0.6

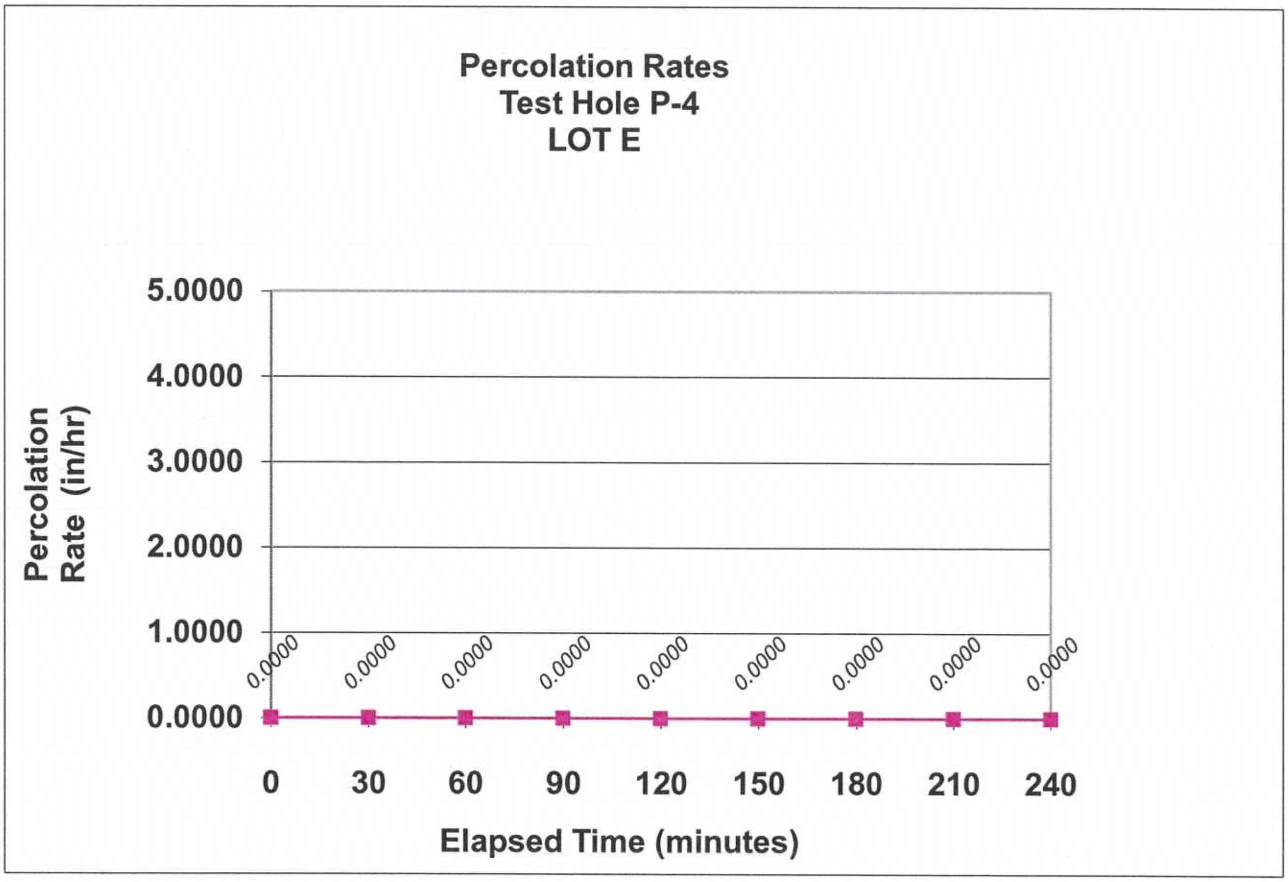


Percolation Test Results For Vallemar Street and Jullaina APN 037-086-027
Project No: SM10391.2
Date: 31 APRIL 2016
By: Haro, Kasunich and Associates

HOLE NO.:	P-4		TEST DATE:3/31/16 DRILL DATE: 3/30/16			
WATER LEVEL AFTER PRE-SOAK:	1.958		DEPTH OF BORING (feet) 2.958			
TESTED BY:	JD		PERCOLATION ZONE (feet):		1.958	2.958
READING	ELAPSED TIME (min)	WATER DEPTH (feet)	REFILL TO (feet)	Incremental Change (in.)	PERCOLATION RATE (min/inch)	PERC (in/hr)
Start	0	1.9583	-	-	-	-
1	30	1.9583	0.0000	0.0000	-	0.0000
2	60	1.9583	0.0000	0.0000	-	0.0000
3	90	1.9583	0.0000	0.0000	-	0.0000
4	120	1.9583	0.0000	0.0000	-	0.0000
5	150	1.9583	0.0000	0.0000	-	0.0000
6	180	1.9583	0.0000	0.0000	-	0.0000
7	210	1.9583	0.0000	0.0000	-	0.0000
8	240	1.9583	0.0000	0.0000	-	0.0000

Average Of Reading's (in/hr)= 0.0000

Reported Percolation Rate (in/hr) = 0.0

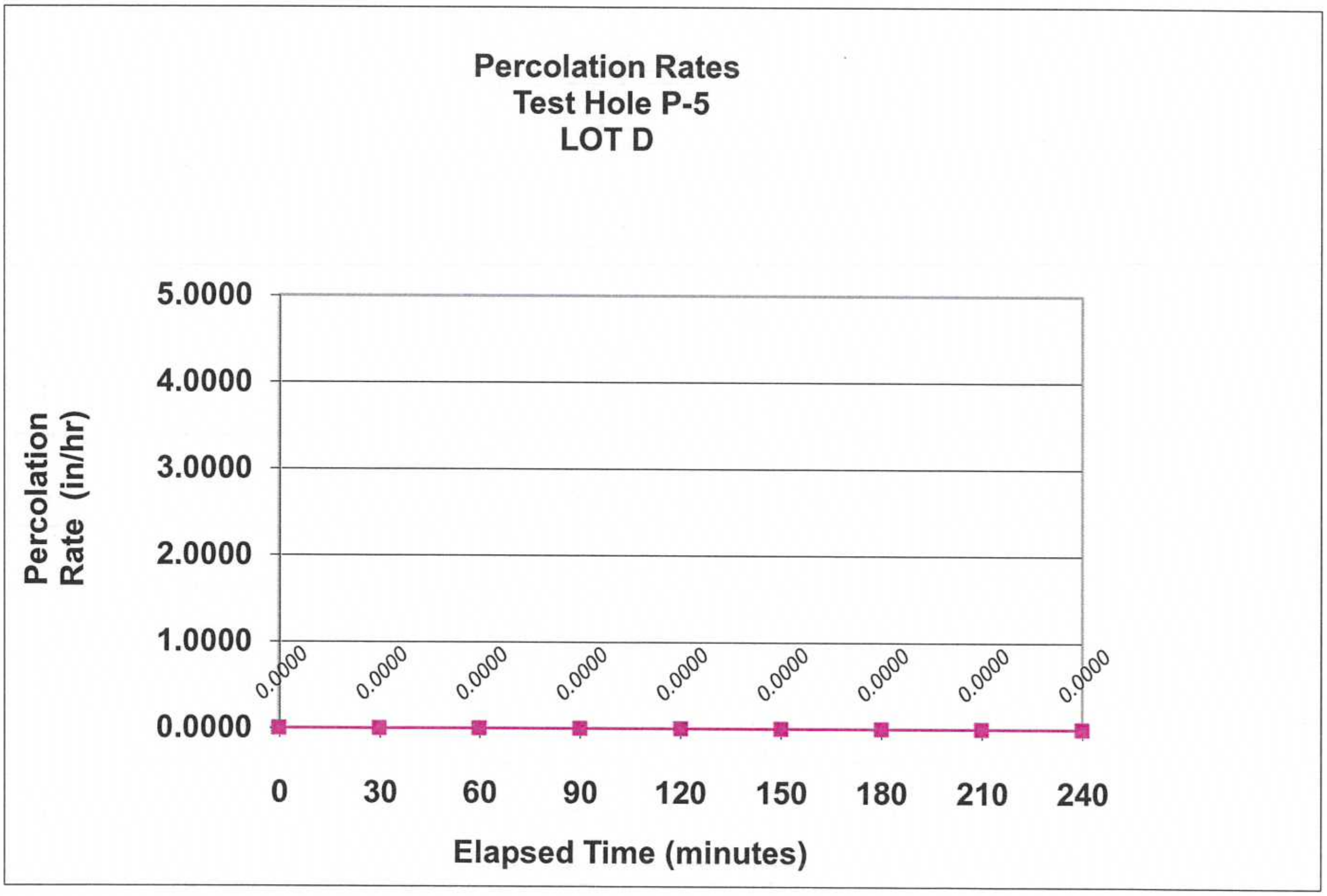


Percolation Test Results For Vallemar Street and Jullaina APN 037-086-026
Project No: SM10391.2
Date: 31 APRIL 2016
By: Haro, Kasunich and Associates

HOLE NO.:	P-4		TEST DATE:3/31/16 DRILL DATE: 3/30/16			
WATER LEVEL AFTER PRE-SOAK: 1.958			DEPTH OF BORING (feet) 2.958			
TESTED BY:	JD		PERCOLATION ZONE (feet):		1.958	2.958
READING	ELAPSED TIME (min)	WATER DEPTH (feet)	REFILL TO (feet)	Incremental Change (in.)	PERCOLATION RATE (min/inch)	PERC (in/hr)
Start	0	1.9583	-	-	-	-
1	30	1.9583	0.0000	0.0000	-	0.0000
2	60	1.9583	0.0000	0.0000	-	0.0000
3	90	1.9583	0.0000	0.0000	-	0.0000
4	120	1.9583	0.0000	0.0000	-	0.0000
5	150	1.9583	0.0000	0.0000	-	0.0000
6	180	1.9583	0.0000	0.0000	-	0.0000
7	210	1.9583	0.0000	0.0000	-	0.0000
8	240	1.9583	0.0000	0.0000	-	0.0000

Average Of Reading's (in/hr)= 0.0000

Reported Percolation Rate (in/hr) = 0.0



APPENDIX E

Scale of Acceptable Risks from Geologic Hazards

APPENDIX E SCALE OF ACCEPTABLE RISKS FROM NON-SEISMIC GEOLOGIC HAZARDS*		
RISK LEVEL	**STRUCTURE TYPE**	**RISK CHARACTERISTICS**
EXTREMELY LOW RISKS	Structures whose continued functioning is critical, or whose failure might be catastrophic: nuclear reactors, large dams, power intently systems, plants manufacturing or storing explosive or toxic materials.	Failure affects substantial populations risk equals nearly zero.
VERY LOW RISKS	Structures whose use is critically needed after a disaster: important utility centers: hospitals: fire, police, and emergency communication facilities; fire stations; and critical transportation elements such as bridges and overpasses; also smaller dams.	Failure affects substantial populations.
LOW RISKS	Structures of high occupancy, or whose use after a disaster: important utility centers; hospitals; fire, police, and emergency communication facilities; fire stations; and critical transportation elements such as bridges and overpasses; also smaller dams.	Failure of a single structure would affect primary only the occupants.
"ORDINARY RISKS"	The vast majority of structures: most commercial and industrial buildings; small hotels and apartment buildings, and single-family residences.	Failure only affects owners/occupants of a structure rather than a substantial population. No significant potential for loss of life of serious physical injury. Risk level is similar or comparable to other ordinary risks (including seismic risks) to citizens of coastal California. No collapse of structures; structural damage limited to repairable damage in most cases. This degree of damage is unlikely as a result of storms with a repeat time of 50 years or less.
MODERATE RISKS	fences, driveways, non-habitable structures, detached retaining walls, sanitary landfills, recreation areas and open space.	Structure is not occupied or occupied infrequently. Low probability of physical injury. Moderate probability of collapse.

***Non-seismic geologic hazards include flooding, landslides, erosion, wave run-up and sinkhole collapse.**

APPENDIX E SCALES OF ACCEPTABLE RISKS FROM SEISMIC GEOLOGIC HAZARDS		
LEVEL OF ACCEPTABLE RISK	**KINDS OF STRUCTURES**	**EXTRA PROJECT COST PROBABLY REQUIRED TO REDUCE RISK TO AN ACCEPTABLE LEVEL**
Extremely Low	Structures whose continued functioning is critical, or whose failure might be catastrophic nuclear reactors, large dams, power intently systems, plants manufacturing or storing explosives to toxic materials.	No set percentage (whatever is required for maximum attainable safety).
Slightly higher than under level 1[1]	Structures whose use is critically needed after a disaster; important utility centers; hospitals; fire, police, and emergency communication facilities; fire station; and critical transportation elements such as bridges and overpasses; also smaller dams.	5 to 25 percent of project cost.
Lowest possible risk to occupants of the structure [3]	Structures of high occupancy or whose use after a disaster would be particularly convenient; schools, churches, theaters, large hotels, and other high-rise buildings housing large numbers of people, other places normally attracting large concentrations of people civic buildings such as fire stations, secondary utility structures, extremely large commercial enterprises, most roads,alternative or non-critical bridges and overpasses.	5 to 15 percent of project cost.
An "ordinary" level or risk to occupants of the structure [3.5]	The vast majority of structures; most commercial and industrial buildings, small hotels and apartment buildings and single-family residences.	1 to 2 percent of project cost in most cases (2 to 10 percent of project cost in a minority of cases)[4]

1. Failure of a single structure may affect substantial populations.

2. These additional percentages are based on the assumption that the base cost is the total cost of the building or other facility when ready for occupancy. In addition, it is assumed that the structure would have been designed and built in accordance with current California practice. Moreover, the estimated additional cost presumes that structures in this acceptable-risk category are to embody sufficient safety to remain functional following an earthquake.

3. Failure of single structure would affect primarily only the occupants.

4. These additional percentages are based on the assumption that the base cost is the total cost of the building or facility when ready for occupancy. In addition, it is assumed that the structures would have been designed and built in accordance with current California Practice. Moreover, the estimated additional cost presumes that structures in this acceptable-risk category are to be sufficiently safe to give reasonable assurance of preventing injury or loss of life during and following an earthquake, but otherwise not necessarily to remain functional.

5. "Ordinary Risk": Resist minor earthquakes without damage; resist moderate earthquakes without structural damage but with some non-structural damage; resist major earthquakes of the intensity or severity of the strongest experienced in California, without collapse, but with some structural, as well as non-structural damage. In most structures, it is expected that structural damage, even in a major earthquake, could be limited to repairable damage. (Structural Engineers Association of California).

Source:<u>Meeting The Earthquake Challenge,</u> Joint Committee on Seismic Safety of the California Legislature, January 1974, p.9.

APPENDIX F

Photographs

Drilling operations at test bore hole B1

Collection of spoils and mixing of grout at test bore hole B1

Grouting of test bore hole B1

Ground surface after drilling operation at test bore hole B1

Drill setup test bore hole B4

Ground surface after drilling operation test bore hole B4

Location of test bore hole B5

Drilling operation test bore hole B5

Grouting test bore hole B2

Drilling operation test bore hole B2

Grouting of test bore hole B3