REPORT
to
MR. CAINE CAMARILLO
MONTEREY PENINSULA REGIONAL PARKS DEPARTMENT
60 GARDEN ROAD, SUITE 325
MONTEREY, CALIFORNIA 93940

GEOTECHNICAL REPORT and COMPUTATIONS for the proposed MECHANICALLY STABILIZED EARTH WALL ROADWAY REPAIR PALO CORONA PARK CORONA ROAD CARMEL, CALIFORNIA

by

GRICE ENGINEERING, INC.
561-A BRUNKEN AVENUE
SALINAS, CALIFORNIA

APRIL 2018

GRICE ENGINEERING INC

ENGINEERING GEOTECHNICS SEPTIC HYDROLOGY
FOUNDATIONS SOILS EARTH STRUCTURES

561A Brunken Avenue
Salinas, California 93901
griceengineering@sbcglobal.net

Salinas: (831) 422-9619
Monterey: (831) 375-1198
FAX: (831) 422-1896

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April 11, 2018

Mr. Caine Camarillo
Monterey Peninsula Regional Parks District
60 Garden Road, Suite 325
Monterey, California 93940

Project: Mechanically Stabilized Earth Wall
Roadway Repair
Palo Corona Park
Corona Road
Carmel, California

Subject: Geotechnical Report and Computations

Dear Mr. Camarillo;

Pursuant to your request, we have completed our geotechnical investigation and evaluation of the above named site. It is our opinion that the proposed repair is suitable for the site, provided the recommendations made herein are followed. Design computations are provided in Appendix C.

In general, the slide occurred due to over saturation of side cast fill materials placed during the construction of back country road. The proposed repair utilizing a Hilfiker Mechanically Stabilized Earth Wall is a suitable method. Recommendations are given relative to this and other characteristics within the report and especially under Special Recommendations.

The report contained herein is made with our best efforts to evaluate the site, determine the site's geotechnical conditions and provide recommendations for these conditions. We submit this report with the understanding that it is the responsibility of the owner, or his representative, to ensure incorporation of these recommendations into the final plans, and their subsequent implementation in the field.

In addition, we recommend that GRICE ENGINEERING, INC., be retained to review the project plans and provide the construction supervision and testing required to document compliance with these recommendations. Should any site condition not mentioned in this report be observed, this office should be notified so that additional recommendations can be made, if necessary.

This report and the recommendations herein are made expressly for the above referenced project and may not be utilized for any other site without written permission of GRICE ENGINEERING, INC.

Please feel free to call this office should you have any questions regarding this report.

Very truly yours,
GRICE ENGINEERING, INC.

REGISTERED PROFESSIONAL ENGINEER
LAWRENCE E. GRICE
RCE 66857
Exp. 09-30-18
041118DIGITAL
CIVIL
STATE OF CALIFORNIA

Lawrence E. Grice, P.E.
R.C.E. 66857

NOTICE TO OWNER

Any earthwork and grading performed without direct engineering supervision and materials testing by Grice Engineering Inc., will not be certified as complete and in accordance with the requirements set forth herein.

Foundations placed without observation of bearing conditions will not be certified as being in accordance with the requirements set forth herein.

Inspection of Work

It is recommended that all site work be inspected and tested during performance by this firm to establish compliance with these recommendations.

NOTIFY:	GRICE ENGINEERING INC.	SALINAS	(831) 422-9619
	561-A Brunken Avenue	MONTEREY	(831) 375-1198
	Salinas, California 93901	FAX	(831) 422-1896

A minimum of 48 hours (2 working days) notification is required prior to commencement of work so that scheduling for testing and inspections can be made.

Please be advised that costs incurred during inspection and testing of all site work is separate and not considered part of the fees as charged by Grice Engineering, Inc. for the report contained herein.

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GEOTECHNICAL REPORT and COMPUTATIONS
for the proposed
MECHANICALLY STABILIZED EARTH WALL
ROADWAY REPAIR - PALO CORONA PARK
CORONA ROAD
CARMEL, CALIFORNIA

Introduction, Method and Scope of Investigation

The purpose of this report is to evaluate the geotechnical properties of the site relative to the design and repair of the existing roadway. From these findings recommendations are given for the design of the development and subsequent construction.

For this purpose, the site was investigated, and prior information concerning construction and subsurface exploration in this area was examined for soils and materials data. The investigation consisted of a detailed site evaluation, which included: a site inspection; a review of literature made available to GRICE ENGINEERING, INC., including Site Plans from Whitson Engineers; geotechnical drilling and soil sampling; materials evaluation; and analysis of the geotechnical properties of the site soils. This report concludes the results of the investigation and provides recommendations based on that work.

The findings and recommendations contained in this report are applicable only to the above named site and its proposed development, and may not be utilized for any other site or purpose without written permission of GRICE ENGINEERING, INC.

Site Description

The project site is located approximately 4.83 miles to the northeast of State Highway One, on Corona Road, in an un-incorporated area of westernmost Monterey County, California. Please refer to the Vicinity and Location Maps and the Site Map in Appendix A for details.

The topography of the site encompasses area containing a moderate to steep downslopes to the east on an east facing hillside, in an area known as Palo Corona, with an elevation of approximately 1,122 feet above mean sea level (msl) in the proposed repair area. The majority of the site is covered with native grasses and trees.

The existing dirt road suffered a slide of the side cast fill forming the outside shoulder during seasonal winter storms. As proposed, the road is to be repaired by construction of a Hilfiker Mechanically Stabilized Earth wall to reconstruct the area of the slide.

Field Investigation

Our field investigation consisted of a site inspection, along with drilling and sampling 1 exploratory bore and inspection of the existing exposures to establish the subsurface soil profile, and obtain sufficient soil specimens to determine the soil characteristics. Drilling was accomplished by hand auger, with the spoil constantly examined, classified, and logged by field method in accordance with the Unified Soil Classification Chart[1] which is the basis of ASTM D2487-10.

Site Soil Profile

As found in the exploratory drilling, the site soils are generally consistent between each of the bores.

Surficial soils are generally fill materials generated on site from the excavated portion of the road or native topsoil. These soils were observed to be a yellowish brown clastic materials composed of fine sands to medium gravel. They contained few amounts of silt and were observed slightly moist and loose.

Weathered granite is located below the topsoil and fill soil. This bedrock was observed to be massive with only few joints.

Complete soil characteristics and comments are reported on the boring logs at the depths observed. The logs are located in Appendix B.

Groundwater

No groundwater was observed at this site to the maximum depth or height of exploration over a 20 feet.

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[1] Adopted 1952 by Corps of Engineers and Bureau of Reclamation. ASTM D2487 was developed as based on the Uniform Soils Classification Chart and System. The methods are equivalent.

Laboratory Testing

Laboratory testing consisted of establishing the *in-situ* ** moisture content and dry density (ASTM D 2487-10) and unconfined penetration, direct shear testing (ASTM D 3080-04). Standard Penetration Resistance values gained during the exploratory drilling are also included.

The following is a tabulation of the field and laboratory test result extremes:

TABLE 1

SUMMARY OF SOIL PROPERTIES

TEST	MAXIMUM	MINIMUM
Unconfined Compression*	9+ kips/ft^2	9+ kips/ft^2
In-Situ Density	120.6 lbs/ft^3	119.9lbs/ft^3
In-Situ Moisture	7.8%	5.4%

All data obtained is reported in Appendix B including the boring logs, with soil classified described at depth observed.

* Pocket Penetrometer

** *In-situ* refers to the in-place state.

Seismic History

Although no fault traces are thought to directly cross the building site, Monterey County is traversed by a number of faults most of which are relatively minor hazards for the purposes of the site development. As such, this site will experience seismic activity of various magnitudes emanating from one or more of the numerous faults in the region.

Various maps presently exist, allowing observation on the site of distinctive geologic features. Some maps, such as that by Burkland and Associates (Reference No. 10) developed for Monterey County, are compilations from various sources detailing the locations of studied faults. Faults have inherit variances within their zones, and discoveries of new fault segments or entire faults is ongoing. There is also some difference in exact fault line location from source map to map, making precise location of said faults difficult. Therefore, relative to the information contained within this report, the following is considered to be as accurate as is currently possible from information made available to Grice Engineering Inc..

Regional Faults

Of most concern are active faults which have tectonic movement in the last 11,000 years and as such are called Holocene Faults and potentially active faults. The following are those nearest listed (Reference No. 12).

The most active is the San Andreas Rift System (Creeping Segment), located approximately 31.9 miles to the northeast. It has the greatest potential for seismic activity with estimated intensities of V-VI Mercalli in this location.

Other fault zones are the San Gregorio-Palo Colorado (Sur) Fault Zone, the center of which is located approximately 3.6 miles to the southwest, the Rinconada Fault Zone, approximately 15.0 miles to the northeast, the Monterey Bay-Tularcitos Fault Zone, approximately 6.1 miles to the northeast, and the Zayante-Vergeles Fault Zone, approximately 27.6 miles to the northeast. These zones are not as liable to rupture as the San Andreas and a seismic event at any of the above fault zones would likely produce earth movements of a lesser intensity at the site.

Liquefaction

The site soils are considered not susceptible to liquefaction as they are typically un-saturated. Granite bedrock is located at a shallow depth.

Differential-Total Settlement - Static and Dynamic

Recommendations given in this Geotechnical Report were developed to reduce the potential for settlement, which is moderate due to the loose sands between grade and approximately ten feet. As the proposed structure is to be supported on weathered granite or other sound material the total and differential settlement is expected to be minimal.

Hydro-Collapse and Subsidence

As observed the near surface soils up to approximate depths of ten feet are loose. These soils possess some capacity to settle under hydraulic loading. However this effect is not common in the area. The recommendations given in this report were established to reduce the potential of this occurring.

The area is not within a known Subsidence Zone.

Slope Stability

Inspection of the site indicates that no landslides are located above or below the construction area and the area is generally not susceptible to slope failure other than loose of loose surficial topsoil and the un-engineered fill materials forming the outside road shoulder.

Slope Stability and Erosion

The parcel was evaluated for landslides located above or below the building area. The site evaluation included the method as delineated in “Special Publication 117A Guidelines for Evaluating and Mitigating Seismic Hazards in California” was reviewed as applicable to this site. The following summarizes the findings.

The following methods and publications were utilized to determine the presence of land movement or excessive erosion above and below the project site.

A. On site evaluation of land features.
B. Aerial photographs spanning the time frame from December 30, 1993 to February 04, 2018.
C. Open File Report 7-718, 1977, Green
D. Geologic Map of California - Santa Cruz Sheet, 1958, Jennings etc.
E. Ground Failures in the Monterey Bay Counties Region, Professional Paper 993, Dept. of the Interior.

1. "Are existing landslides, active or inactive, present on, or adjacent (either uphill or downhill) to the project site?"

Review of the terrain around the site and aerial images indicates that the natural terrain is generally free from markings of moderate to deep seated slides however solifluction is common in the surficial soils on steeper slopes. Most erosion or loss of material is associated with drainage from the local earth roads

The generally area is considered not susceptible to mass slope failure due to the presence of granite bedrock at a shallow depth.

No features or conditions were visually observed during the site exploration which indicate or suggest landsliding beyond solifluction has or will occur above or below the project site.

No recorded features were noted on any of the reviewed publications which suggest, imply or note landslides have or will occur above or below the project site.

2. "Are there geologic formations or other earth materials located on or adjacent to the site that are known to be susceptible to landslides?"

There are no geologic formations, or other earth materials located on or adjacent to the site that is known to be susceptible to landslides other than that discussed above.

3. "Do slope areas show surface manifestations of the presence of subsurface water (springs and seeps), or can potential pathways or sources of concentrated water infiltration be identified on or upslope of the site?"

No springs or seeps or the indication of such were observed during the site exploration. Review of the aerial imagery did not indicate any locations of seepage as suggested by increased or more active vegetation or topography (erosion scarp, slump). Spring or seeps within the general area and lithology are occasionally present.

Drainage over the local terrain is unfocused.

Inspection of areal photographs spanning from December 30, 1993 to February 04, 2018 indicates the terrain and presence of vegetation has been consistent during that period. Construction of the existing road predates these images.

These characteristics in conjunction with the firm soils indicated a low potential for rapid solifluction or debris flow.

4. “Are susceptible land forms and vulnerable locations preset?”

No excessively steep or erodible slopes are located above or below the site.

5. “Given the proposed development, could anticipated changes in the surface and subsurface hydrology (due to watering of lawns, on-site sewage disposal, concentrated runoff from impervious surfaces, etc.) increase the potential for future landsliding in some areas?”

The area is generally not open for further development.

Seismic Strength Loss

The site soils are considered resistant to seismic strength loss and the resulting momentary liquefaction. The relatively short duration of earthquake loading will not provide a significant number of high amplitude stress cycles to alter the strain characteristics. Additionally the clay-silt fraction is not considered quick nor sensitive, as such it will not have the associated loss of strength.

Chemical Reactivity

Generally the local bedrock and soils generated from it are not known for sulfate reaction with Portland cement products and as such chemical reactivity is not considered a problem in this area.

Expansive Soils

In general the site soils are or contain silty sands of very low plasticity. These soils are typical to the area. Additionally there are no known problems with expansive soils in the area.

Surface Rupture and Lateral Spreading

The project site is located 3.6 miles to the northeast of the Palo Colorado Fault (as part of the San Gregorio, Sur Section, Fault Zone, inferred, concealed beneath the Pacific Ocean, Holocene). The site inspection did not reveal any surface features indicating a fault rupture has occurred at the site. The existing structure, driveways and roads do not reveal any strains which would be attributable to subsurface lateral or vertical displacements resulting from fault slip. Therefore surface rupture from fault activity across the site is considered improbable.

The project site is underlain by relatively strong soils and soft bedrock. These materials are considered resistant to lateral spreading. As such surface rupture from lateral spreading is considered improbable.

Seismicity

It is recommended that all structures be designed and built in accordance with the requirements of the California Building Code's current edition. All buildings should be founded on undisturbed native soils and/or tested and accepted engineering fill to prevent resonance amplification between soils and the structure.

2016 California Building Code Geoseismic Classifications

The California Building Code, 2016 edition (Reference No. 13), provides for seismic design values. These values are to be utilized when evaluating structural elements. The soils profile determination is based on the penetration resistance data developed from advancement of exploratory bores. Using estimated penetration values per depth of soils type gives an overall site value of greater than 50 blows/foot penetration resistance as per Equation 20.4-3, ASCE 7-10. The geoseismic character is as listed in the following table.

2015 I.B.C. - 2016 C.B.C. EARTHQUAKE LOADS: SECTION 1613				
LATITUDE	36.484165	SOIL PROFILE:	Very Dense Soil / Soft Bedrock	
LONGITUDE	-121.886165	SITE CLASS	C	
PERIOD	S	F	Sm	Sd
0.2 sec	Ss = 1.159	Fa = 1.0	Sms = 1.159	Sds = 0.773
1.0 sec	S1 = 0.424	Fv = 1.0	Sm1 = 0.584	Sd1 = 0.584
Seismic Design Category to be assigned by structural engineer or designer				

CONCLUSIONS OF INVESTIGATION

Special Recommendations

Review of the site characteristics and extent of the slide indicate that the most applicable method of repair is use of Mechanically Stabilized Earth (MSE) fill. The use of a Hilfiker product is typical in this regard.

To match the existing grades and replace the approximate original surface within the boundary of the slide it is recommended to use the component of Hilfiker Welded Wire Wall. The inclination of wall face can be constructed at the typical slope of 1H:48V or a shallower slope providing the resulting crown in outside of the proposed guardrail and traveled way.

The structure is to be provided with complete back drainage. Such drainage may take the form of couches at rear heal of each 2 foot step or a vertical curtain. Collection of each couch or curtain may be internal or external. A variety of methods are suitable. The collected drainage should be let to grade sufficiently below the wall to reduce concerns of erosion.

As an alternative to back drains the lower mats may be filled with open graded gravel provided with one discharge pipe at the bottom discharging to daylight.

The full width and depth of the MSE is to be embeded into suitable materials which is granite bedrock at this location. This will most likely be readily completed throughout most the structure, however unsuitable topsoil or existing fill may be present at the beginning and end of the wall. As such further recommendations may be given at these locations.

The base of all excavations and over-excavations are to be inspected by the Soils Engineer prior to further processing, steel or form placement.

Any further site activity, especially grading and foundation excavations, should be under the direction of a qualified Soils Engineer or their Representative.

Should the spectrum of development change, this office should be notified so that additional recommendations can be made, if necessary.

Bearing and Retaining Soil Pressures

Geotechnical evaluation of the site earth materials indicates the engineered fill and topsoil is unsuitable for support of foundations or on grade structures without being processed as engineered fill.

The native granite bedrock is preferable support for the proposed M.S.E. wall and any other structure. The allowable bearing for dead plus live loads is 4.0 kips per square foot. Adjacency to slopes may require other recommendations for this condition.

The weathered to unweathered granite bedrock provides a high to very high bearing capacity, generally beyond the needs of this type of structure. As such higher values may be available if required.

Embedment depths do not take into account the loose upper top soils, disturbed soils or any other unacceptable soils which exist at the site, e.g., any un-engineered fill, landscaping soils, etc. One-third increase to be allowed for wind and seismic forces.

Lateral Soil Pressures

Lateral pressures exterior to the M.S.E. structures are mostly the resultant of supporting weathered granite. As the granite is generally self supporting actual lateral pressures are of low concern. However to provide for the potential presence of earth (engineered fill or native) behind the wall a modest value was applied in the wall stability computations. To review the values please refer to the M.S.E. computations in Appendix C.

Slope Ratio and Drainage

Analysis of site soil and rock indicate that cut slope ratio of exposed granite can vertical. Cut slopes exposing the native soils should be inclined approximately 1H:1V or less where the accumulation of sloughed materials are of concern otherwise they can be vertical and will eventually recline to a position of stability. This condition will result in the accumulation of sloughed materials at the toe of the slope.

Un-reinforced fill (e.g. not mechanically stabilized) slopes ratios of 2 horizontal to 1 vertical will be satisfactory provided they are landscaped with soil retaining ground covers and are protected against concentrated over slope drainage.

The face of Mechanically Stabilized Earth can be constructed up to that recommended by the component manufacturer unless otherwise determined.

Surface Drainage and Erosion Control

Design and construction of the project should fit the topographic and hydrologic features of the site. It is important to minimize unnecessary grading of or near steep slopes. Disturbing native vegetation and natural soil structure allows runoff velocity and transport of sediments to increase.

General surface drainage should be retained at low velocity by slope, sod or other energy reducing features sufficient to prevent erosion, with concentrated over-slope drainage carried in lined channels, flumes, pipe or other erosion-preventing installations.

Runoff flows should be directed into pipes or lined ditches and then onto an energy dissipater before discharging into streams or drainage ways. De-silting should be provided as necessary and may take form of stilling basins, gravel berms, forested/vegetated screens, etc.

All concentrated roof and area drainage should be conveyed and released to grade below the road structure and to a suitable point of release. The actual outlet of concentrated drainage should be reviewed in the field. Subsurface dispersal is not allowed for this project.

During construction, never store cut and fill material where it may wash into streams or drainage ways. Keep all culverts and drainage facilities free of silt and debris. Keep emergency erosion control materials such as straw mulch, plastic sheeting, and sandbags on-site and install these at the end of each day as necessary.

Re-vegetate and protect exposed soils by October 15. Use appropriate grass/legume seed mixes and/or straw mulch for temporary cover. Plan permanent vegetation to include native and drought tolerant plants. Seeding and re-vegetation may require special soil preparation, fertilizing, irrigation, and mulching.

Subsurface Drains

Use of spun filter fabric is not recommended for use in construction subsurface drains collecting seepage as this type of fabric typically becomes clogged. Should filter fabric be necessary it is recommended that a woven fabric be used such as Mirafi Filterweave 300. Otherwise we would recommend omission of the fabric and placement of Caltrans Class 1, Type ‘A” or “B” drain rock, and that any fabric only be placed near the top of the trench between the gravel and earth backfill or where the gravel extends to grade, 1 foot below finish grade.

CLASS 1		
SIEVE SIZES	**PERCENTAGE PASSING**	
	TYPE A	TYPE B
50.0-mm/2 inches	----	100
37.5-mm/1.5 inches	----	95-100
19.0-mm/0.75 inches	100	50-100
12.5-mm/0.5 inches	95-100	-----
9.5-mm/0.415 inches	70-100	15-55
4.75-mm/No. 4	0-55	0-25
2.36-mm/No. 8	0-10	0-5
75.0-µm/No.200	0-3	0-3

General Grading Recommendations

For those items not directly addressed, it is recommended that all earthwork be performed in accordance with the following.

General: This item shall consist of all clearing and grubbing; preparation of land to be filled; excavation and fill of the land; spreading, compaction and control of the fill; and all subsidiary work necessary to complete the graded area to conform with the lines, grades and slopes as shown on the approved plans.

The Contractor shall provide all equipment and labor necessary to complete the work as specified herein, as shown on the approved plans as stated in the project specifications.

Preparation: Site preparation will consist of clearing and grubbing any existing structures and deleterious materials from the site, and the earthwork required to shape the site to receive the intended improvements, in accordance with the recommended grading specifications and the recommendations as provided above.

All vegetable matter, irreducible material greater than 4 inches and other deleterious materials shall be removed from the areas in which grading is to be done. Such materials not suitable for reuse shall be disposed of as directed.

After the foundation for fill has been cleared, it shall be brought to the proper moisture content by adding water or aerating and compacting to a Relative Compaction of not less than 90% or as specified. The soils shall be tested to a depth sufficient to determine quality and shall be approved by the Soils Engineer for foundation purposes prior to placing engineered fill.

General Fill: General fill shall be placed only on approved surfaces, as engineered fill, and shall be compacted to 90% Relative Compaction. Native soils accepted for fill or existing aggregate fill may be used for fill purposes provided all aggregate larger than 6 inches are removed. The material for engineered fill shall be approved by the Soils Engineer before commencement of grading operations.

Each layer shall be compacted to a Relative Compaction of not less than 90% or as specified in the soils report and on the accepted plans. Compaction shall be continuous over the entire area of each layer.

The selected fill material shall be placed in layers which, when compacted, shall not exceed 6 inches in thickness. Each layer shall be spread evenly and shall be thoroughly mixed during the spreading to ensure uniformity of material in each layer. Fill shall be placed such that cross fall does not exceed 1 foot in 20 unless otherwise directed.

When fill material includes rock or concrete rubble, no irreducible material larger than 4 inches in greatest dimension will be allowed except under the direction of the Soils Engineer.

<u>Imported Materials:</u> Materials imported for fill purposes shall be classified as: SAND, group symbol SW, SP, SC or SM, as given in ASTM 2487-10, "The Classification of Soils For Engineering Purposes." In all cases the portion finer than the No. 200 sieve shall not contain any greatly expansive clays and shall be free from vegetable matter and other deleterious materials. The material for engineered fill shall be approved by the Soils Engineer before commencement of grading operations.

<u>Structural Backfill:</u> Trench, wall and structural backfill shall be placed only on approved surfaces, as engineered fill, and shall be compacted to 95% Relative Compaction. Materials imported for backfill purposes shall have a Sand Equivalent of no less than 30 and shall be classified as Clean Sands as designated in "The Classification of Soils For Engineering Purposes" (ASTM 2487-10).

<u>Pavement Grades:</u> All pavement grades shall be of uniform thickness, density and moisture prior to placement of the next grade. Flexure of each or all grades shall not exceed 0.25 inches in 5 feet under an axial load of 18.5 kip.

<u>Aggregate Base Course:</u> All aggregates used for specified base courses, shall be handled in a manner which prevents segregation and non-uniformity of gradation.

<u>Compaction:</u> All re-compacted soils and/or engineered fill should be placed at a minimum 90% Relative Compaction or at the value required for that portion of the work. All pavement sections should be compacted to a minimum of 95% Relative Compaction.

Field density testing shall be completed by the Soils Engineer on each compacted layer or as determined by the Soils Engineer. At least one test shall be made for each 500 cubic yards or fraction thereof, placed with a minimum of two tests per layer in isolated areas. Where a sheeps'-foot roller is used, the soil may be disturbed to a depth of several inches. Density tests shall be taken in compacted materials below the disturbed surface. When these tests indicate that the density of any layer of fill or portion thereof, is below the required density, that particular layer or portion shall be reworked until the required density has been obtained.

<u>Moisture:</u> During compaction moisture content of native soils should be that consistent with the moisture relative to 95% Relative Compaction and in no case should these materials be placed at less than 3 percent above the specific optimum moisture content for the soil in question. The engineer may elect to accept high moisture compacted soils provided the materials are at 95% Relative Wet Density at that moisture content.

The moisture content of the fill material shall be maintained in a suitable range to permit efficient compaction. The Soils Engineer may require adding moisture, aerating, or blending of wet and dry soils.

All earth moving and work operations shall be controlled to prevent water from running into and pooling in excavated areas. All such water shall be promptly removed and the site kept drained.

<u>Tests:</u> All materials placed should be tested in accordance with the Compaction Control Tests: "Density of Soil In-Place by Sand Cone Method" (ASTM D-1556-07), "Moisture-Density Relationship of Soils" (ASTM D-1557-09), and "Density of Soils In-Place by Nuclear Method" (ASTM D-6938-10).

The standard test used to define maximum densities of all compaction work shall be the A.S.T.M. D-1557-09, Moisture Density of Soils, using a 10-pound ram and 18-inch drop. All densities shall be expressed as a relative density in terms of the maximum density obtained in the laboratory by the foregoing standard procedure.

<u>Deleterious Materials:</u> Materials containing an excess of 5% (by weight) of vegetative or other deleterious matter may be utilized in areas of landscaping or other non-structural fills. Deleterious material includes all vegetative and non-mineral material, and all non-reducible stone, rubble and/or mineral matter of greater than 6 inches.

<u>Over-Excavations:</u> Over-excavations, when required, should include the foundation and pavement envelopes. Such excavations should extend beyond edge of development a minimum of 5 feet and to an imaginary line extending away and downward at a slope of 45 degrees from the edge of development. The process shall include the complete removal of the required soils and subsequent placement of engineered fill. After removal of the soils to the

required depth, the base of the excavation shall be inspected and approved by the Soils Engineer or his representative prior to further soils processing or placement. Based on this inspection other recommendations may be made.

<u>Existing Conditions:</u> In developed areas underground utilities may be located within the area of proposed construction. In addition, buried objects or deeply disturbed soils may also be encountered. As such all care and practice is to be exercised to observe for and locate any such objects. Where these objects are to be removed or use discontinued, they are to be removed in their entirety and all disturbed soils are to be processed as engineered fill.

<u>Key:</u> All fills on slopes greater than 1 vertical to 6 horizontal shall be keyed into the adjacent soil. The toe of all slopes should be supported by a key cut a minimum of 3 feet into undisturbed soils to the inside of the fills toe. This key should be a minimum of 6 feet in width and slope at no less than 10% into the slope. In addition, as the fill advances up slope benches, 3 feet across, should be scarified into the fill/undisturbed soil interface.

<u>Seasonal Limits:</u> When the work is interrupted by rain, fill operations shall not be resumed until field tests by the Soils Engineer indicate that the moisture content and density of the fill is as previously specified and soils to be placed are in suitable condition

<u>Unusual Conditions:</u> In the event that any unusual conditions are encountered during grading operations which are not covered by the soil investigation or the specifications, the Soils Engineer shall be immediately notified such that additional recommendations may be made.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

The recommendations of this report are based on our understanding of the project as represented by the plans, and the assumption that the soil conditions do not deviate from those represented in this site soils investigation. Therefore, should any variations or undesirable conditions be encountered during construction, or if the actual project will differ from that planned at this time, GRICE ENGINEERING INC. should be notified and provided the opportunity to make addendum recommendations if required.

NOTIFY:	GRICE ENGINEERING INC.	SALINAS	(831) 422-9619
	561-A Brunken Avenue	MONTEREY	(831) 375-1198
	Salinas, California 93901	FAX	(831) 422-1896

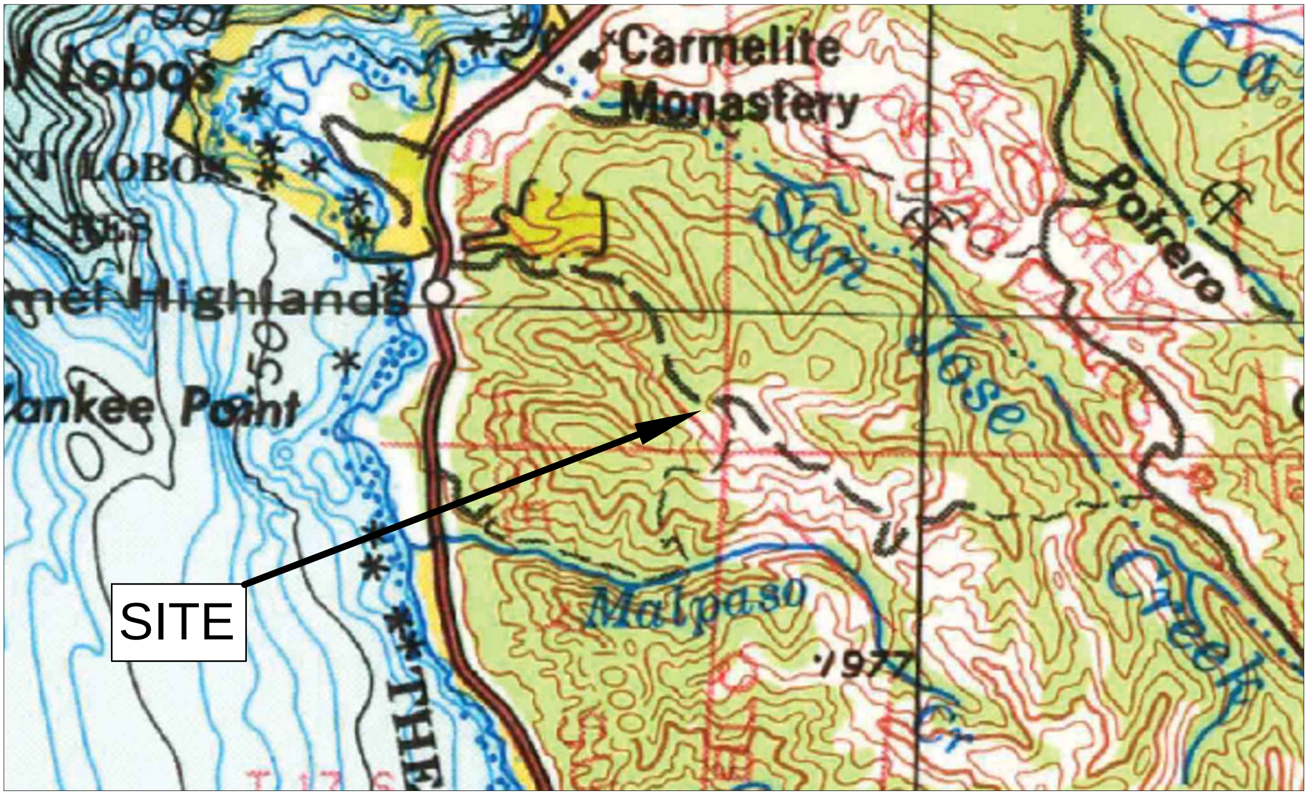
This report is issued with admonishment to the Owner and to his representative(s), that the information contained herein should be made available to the responsible project personnel including the architects, engineers, and contractors for the project. The recommendations contained herein should be incorporated into the plans, the specifications, and the final work.

It is requested that GRICE ENGINEERING INC. be retained to review the project grading and foundation plans to ensure compliance with these recommendations. Further, it is the position of GRICE ENGINEERING INC. that work performed without our knowledge and supervision, or the direction and supervision of a project responsible professional soils engineer renders this report invalid.

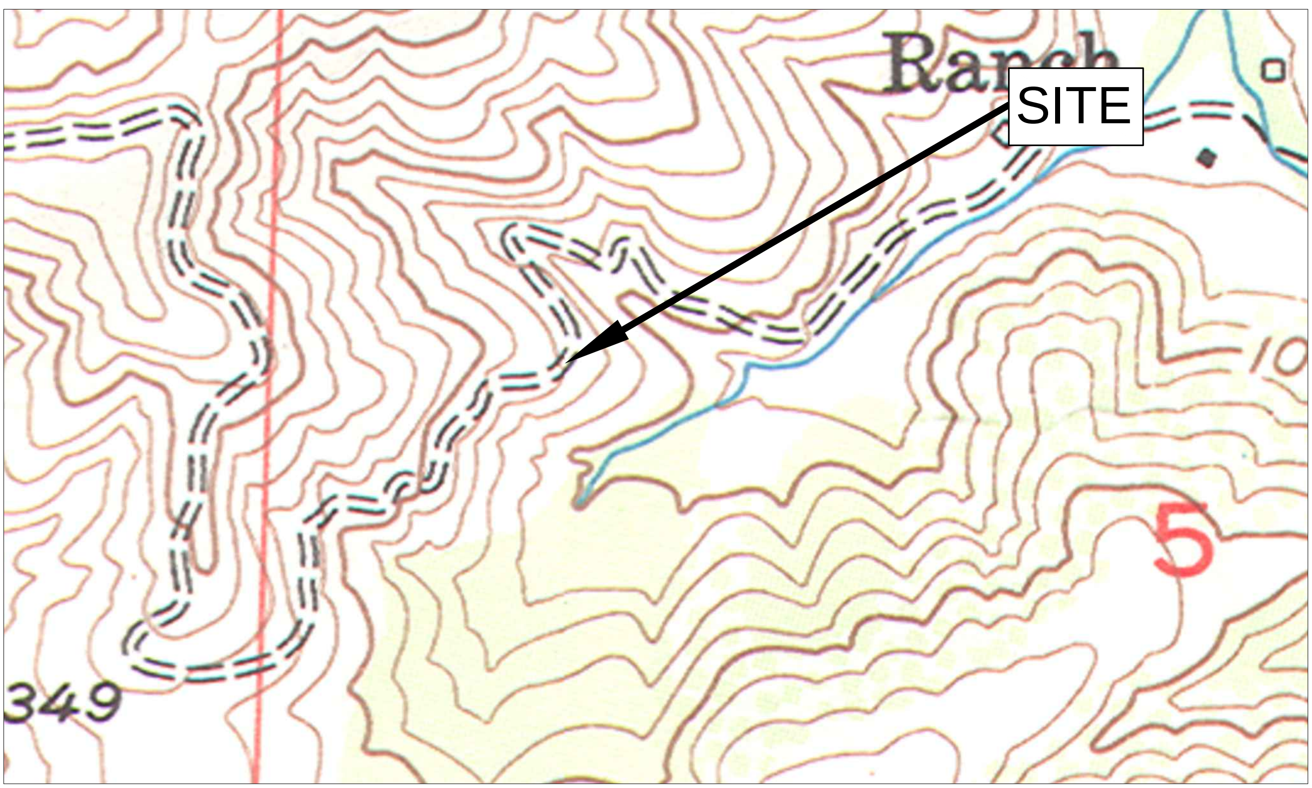
It is our opinion the findings of this report are **valid** as of the **present date**, **<u>however</u>**, changes in the **Codes and Requirements** can occur and change the recommendations given within this report concerning the property. In addition changes in the conditions of a property can occur with the passage of time, due either to natural processes or to the works of man and may effect this property. In addition, changes in **standards** may occur as a result of legislation, or the broadening of knowledge, and these changes may require re-evaluation of the conditions stated herein. Accordingly, the findings of this report may be invalidated wholly, or partially, by changes beyond our control. Therefore, this report is subject to review and should not be relied upon after a period of **<u>three years</u>**.

REVISED 01-07-2011

APPENDIX A



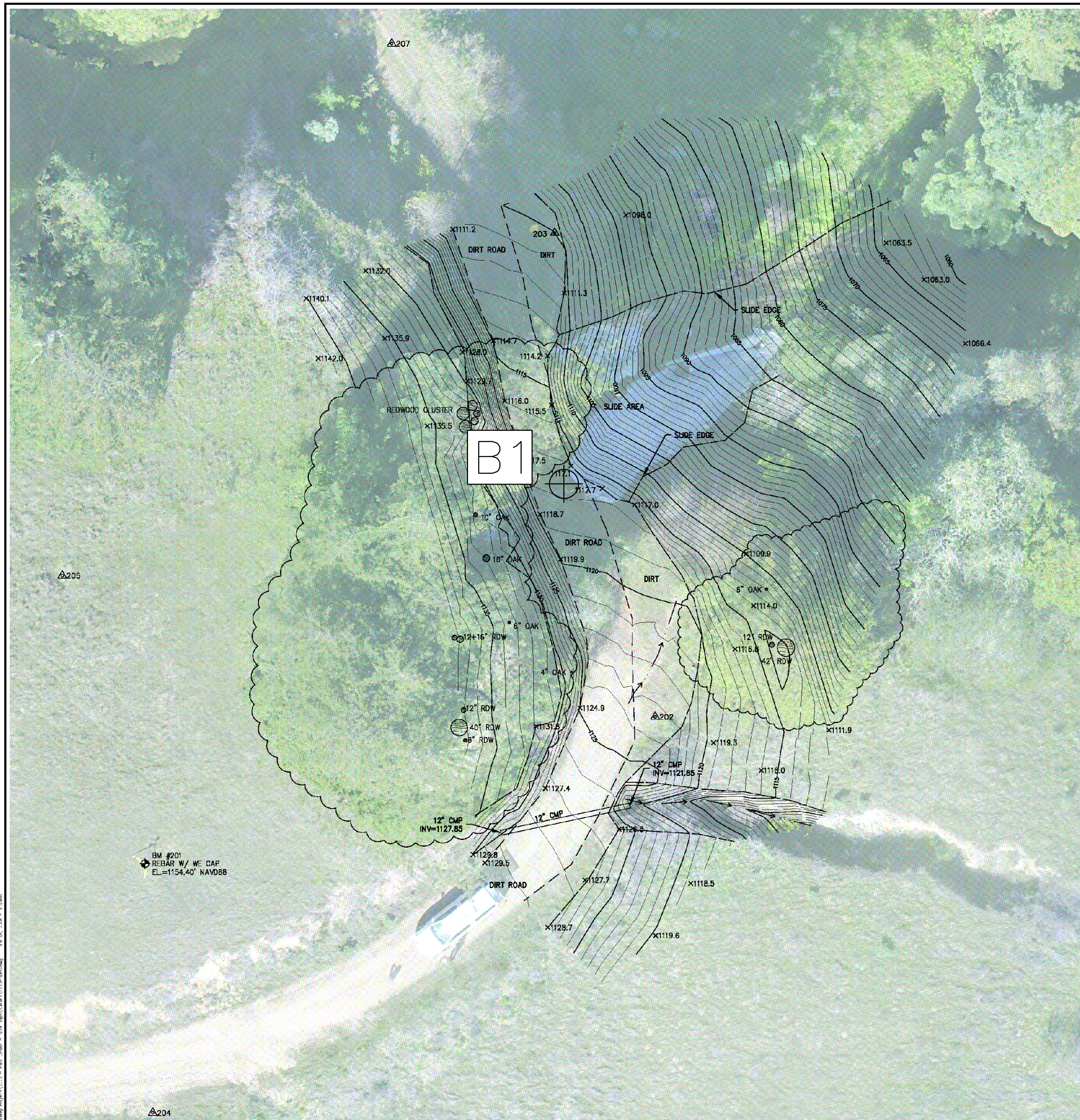
Vicinity Map



Location Map

GRICE ENGINEERING INC
561A Brunken Avenue
Salinas, California 93901
Salinas: (831) 422-9619
Monterey: (831) 375-1198 FAX: (831) 422-1896

Vicinity and Location Map
Road Repair - Palo Corona Park -Corona Road
Carmel, CA File No. 6943-18.04



NOTES:

1. THIS MAP REPRESENTS A TOPOGRAPHIC SURVEY PERFORMED BY WHITSON ENGINEERS ON FEBRUARY 28, 2018.
2. THIS MAP PORTRAYS THE SITE AT THE TIME OF THE SURVEY AND DOES NOT SHOW SOILS OR GEOLOGY INFORMATION, UNDERGROUND CONDITIONS, EASEMENTS, ZONING OR REGULATORY INFORMATION OR ANY OTHER ITEMS NOT SPECIFICALLY REQUESTED BY THE CLIENT.
3. DISTANCES AND DIMENSIONS SHOWN ARE EXPRESSED IN FEET AND DECIMALS THEREOF, UNLESS OTHERWISE NOTED.
4. BENCHMARK TAKEN AS A 1/2" REBAR WITH PLASTIC CAP STAMPED "WHITSON CONTROL", DESIGNATED POINT NO. 201, SHOWN HEREON. ELEVATION: 1154.40' NAVD88, AS DETERMINED THROUGH STATIC GPS OBSERVATIONS AND THE USE OF THE NATIONAL GEODETIC SURVEY'S ONLINE POSITIONING USER SERVICE (OPUS)
5. UNDERGROUND UTILITIES, IF SHOWN, ARE BASED ON FIELD LOCATION OF VISIBLE SURFACE FEATURES ONLY. EXACT LOCATIONS AND DEPTHS SHOULD BE CONFIRMED PRIOR TO CONSTRUCTION.
6. DIAMETERS OF TREES ARE SHOWN IN INCHES MEASURED AT 2 FEET ABOVE GRADE. TREES SMALLER THAN 6" WERE NOT NECESSARILY LOCATED AS PART OF THIS SURVEY.

Know what's below. Call before you dig. 811

LEGEND

100	GROUND CONTOUR
200	CONTROL POINT
BM	BENCHMARK
X 1100.00	SPOT GRADE
12" RDW	12" DIAMETER REDWOOD TREE
	TREE DRIP LINE
	FLOW LINE
CMP	CORRUGATED METAL PIPE

10 0 10 20 30 Feet

SCALE: 1" = 10'

N

REVISIONS:

NO.	BY:	DATE:	DESCRIPTION:

DATE: 4/3/18
SCALE: 1"=10'
ENGR. MVH
JOB NO.: 3773.00

LICENSED LAND SURVEYOR RICHARD P. WEBER No. 8002 STATE OF CALIFORNIA

WHITSON ENGINEERS
6 Harris Court · Monterey, CA 93940
831 649-5225 · Fax 831 373-5065
CIVIL ENGINEERING · LAND SURVEYING · PROJECT MANAGEMENT

PALO CORONA SLIDE REPAIR
MONTEREY COUNTY CALIFORNIA
TOPOGRAPHIC SURVEY
EXISTING CONDITIONS
APN 417-011-033

SHEET 1 OF 1

0 1 2 INCHES 3

APPENDIX B

Palo Corona, Palo Corona Road, Carmel

Boring No. 1

March 22, 2018

Depth	Symbol	Sample	Blow Count per 6 inch	Description	Auger Pen.	Density	Moisture	Unconfined	Cohesion	Shear
1.00				(CUTTINGS) Yellowish brown | CLASTS; very fine sand to medium gravel | few: silt | slightly moist; soft, loose.						
1.00										
				Density and moisture content determined by nuclear densitometer-hygrometer		119.9	7.8	9+		
2.00										
				End of bore at 2.1, refusal in rock.		120.6	5.4	9+		
				Depth of fill/topsoil is generally 0 to 10 feet with the deepest portion under the crown of the outside shoulder.						
3.00										
4.00										
5.00										
				Below the fill or topsoil (INPLACE) Speckled; yellowish brown, pale yellow , very dark gray | WEATHERED GRANITE; dense; lightly fractured or jointed | (CUTTINGS) SANDS; fine to coarse; occasional to medium gravel; angular | trace to few (dependent on weathering): silt | slightly moist; hard						
6.00										
7.00										
				Bore located at head of scarp.						
				Inspection of existing road cut and slope exposed by slide indicate similar lithology						
				Total exposure including cut and slide banks is over 20 feet.						
8.00										
9.00										
10.00										
11.00										
12.00										
13.00										
14.00										
15.00										
16.00										
17.00										
18.00										
19.00										
20.00										

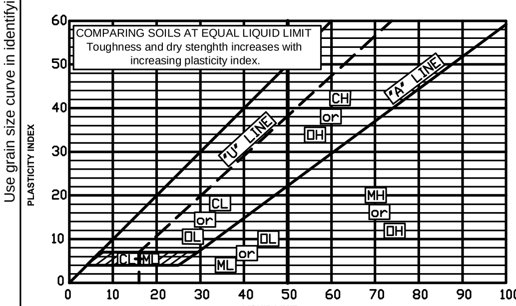
UNIFIED SOIL CLASSIFICATION & ASTM D2487: INCLUDING IDENTIFICATION AND DESCRIPTION

Major Divisions			FIELD IDENTIFICATION PROCEDURES Excluding particles larger than 3 inches and basing fractions on estimated weights	GROUP SYMBOLS N.	TYPICAL NAMES	INFORMATION REQUIRED FOR DESCRIBING SOILS	LABORATORY CLASSIFICATION CRITERIA
COARSE GRAINED SOILS More than half of material is **larger** than No. 200 sieve size is (The No. 200 sieve size is about the smallest particle visible to the naked eye)	GRAVELS More than half of coarse fraction is larger than No. 4 sieve size (For visual classifications, the 1/4" size may be used as equivalent to the No. 4 sieve size.)	CLEAN GRAVELS (Little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes.	GW	Well graded gravels, gravel-sand mixtures, little or no fines.	Give typical name, indicate approximate percentages of sand and gravel, max. size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information, and symbol in parentheses.	Determine percentages of gravel and sand from grain size curve. Depending on percentage of fines (fractin smaller than No. 200 sieve size) coarse grained soils are classified as follows: Less than 5% GW,GP, SW, SP; More than 12 % GM, GC, SW, SC; 5% to 12% **Borderline** cases requiring use of dual symbols. — $C_u = \frac{D_{60}}{D_{10}}$ Greater than 4; $C_c = \frac{(D_{30})^2}{(D_{10} \times D_{60})}$ Between one and 3
			Predominatly one size or a range of sizes with some intermediate sizes missing.	GP	Poorly graded gravels, gravel-sand mixtures, little or no fines.		Not meeting all gradation requirements for GW
		GRAVELS WITH FINES (Appreciable amount of fines)	Non-plastic fines (for identification procedures see ML below).	GM	Silty gravels, poorly graded gravel-sand-silt mixtures.		Atterberg limits below "A" line or PI less than 4 — Above "A" line with PI between 4 and 7 are **borderline** cases requiring use of dual symbols
			Plastic fines (for identification procedures see CL below).	GC	Clayey gravels, poorly graded gravel-sand-clay mixtures.	For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics.	Atterberg limits above "A" line or PI greater than 7
	SANDS More than half of coarse fraction is smaller than No. 4 sieve size	CLEAN SANDS (Little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate particle sizes.	SW	Well graded sands, gravelly sands, little or no fines.		$C_u = \frac{D_{60}}{D_{10}}$ Greater than 6; $C_c = \frac{(D_{30})^2}{(D_{10} \times D_{60})}$ Between one and 3
			Predominatly one size or a range of sizes with some intermediate sizes missing.	SP	Poorly graded sands, gravelly sands, little or no fines.	EXAMPLE:	Not meeting all gradation requirements for SW
		SANDS WITH FINES (Appreciable amount of fines)	Non-plastic fines (for idendification procedures see ML below).	SM	Silty sands, poorly graded sand-silt mixtures.	**Silty Sand,** gravelly; about 20% hard, angular gravel particles $\frac{1}{2}$ inch maximum size; rounded and subangular sand grains coarse to fine, about 15 % non-plastic fines with low dry strength, well compacted and moist in place, alluvial sand; (SM).	Atterberg limits below "A" line or PI less than 4 — Above "A" line with PI between 4 and 7 are **borderline** cases requiring use of dual symbols
			Plastic fines (for identification procedures see CL below).	SC	Clayey sands, poorly graded sand-clay mixtures.		Atterberg limits above "A" line or PI greater than 7

IDENTIFICATION PROCEDURES ON FRACTION SMALLER THAN No. 40 SIEVE SIZE

Major Divisions		DRY STRENGTH (CRUSHING CHARACTERISTICS)	DILATANCY (REACTION TO SHAKING)	TOUGHNESS (CONSISTENCY NEAR PLASTIC LIMIT)	GROUP SYMBOLS N.	TYPICAL NAMES	INFORMATION REQUIRED FOR DESCRIBING SOILS
FINE GRAINED SOILS More than half of material is **smaller** than No. 200 sieve size (The No. 200 sieve size is about the smallest particle visible to the naked eye)	SILTS AND CLAYS Liquid limit less than 50	None to slight	Quick to slow	None	ML	Inorganic silts and very vine sands, rock flour, silty or clayey fine sands withg slight plasticity.	Give typical name, indicate degree and character of plasticity, amount and maximum size of coarse grains, color in wet conditions, odor if any, local or geologic name, and other pertinent descriptive information, and symbol in parentheses.
		Medium to high	None to very slow	Medium	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.	
		Slight to medium	Slow	Slight	OL	Organic silts and organic silt-clays of low plasticity.	
	SILTS AND CLAYS Liquid limit greater than 50	Slight to medium	Slow to none	Slight to medium	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.	For undisturbed soils add information or structure, stratification, consistency in undisturbed and remolded states, moisture and drainage conditions.
		High to very high	None	High	CH	Inorganic clays of high plasticity, fat clays.	EXAMPLE:
		Medium to high	None to very slow	Slight to medium	OH	Organic clays of medium to high plasticity.	**Clayey silt**, brown, slightly plastic, small percentage of fine sand, numerous vertical root holes, firm and dry in place, loess; (ML).
HIGHLY ORGANIC SOILS		Readily identified by color, odor, spongy feel and frequently by fibrous texture.			Pt	Peat and other highly organic soils.	

Use grain size curve in identifying the fractions as given under field identification.



PLASTICITY CHART

FOR LABORATORY CLASSIFICATION OF FINE GRAINED SOILS

N. **Boundary** classifications: Soils possessing characteristics of two groups are designated by combinations of group symbols. For examle GW-GC, well graded gravel-sand mixture with clay binder.
N. All sieve sizes on this chart are U.S. Standard.

FIELD IDENTIFICATION PROCEDURES FOR FINE GRAINED SOILS OR FRACTIONS

These procedures are to be performed on the minus No. 40 sieve size particles, approximately $\frac{1}{64}$ inches. For field classification purposes, screening is not intended; simply remove by hands the coarse particles that interfere with the test.

DILATANCY (Reaction to shaking)

After removing portions larger than No. 40 sieve size, prepare a pot of moist soil with a volume of about one-half cubic inch. Add enough water if necessary to make the soil soft but not sticky.

Place the pot in the open palm of one hand and shake horizontally, stricking vigorously against the other hand several times. A positive reaction consists of the appearance of water on the surface of the pot which changes to a livery consistancy and becomes glossy. When the sample is squeezed between the fingers, the water and gloss disappear from the surface, the pot stiffens and finnally it cracks or crumbles. The rapidity of appearance of water during shaking and of its diappearance during squeezing assist in identifying the character of the fines in a soil.

Very fine clean sands give the quickes and most distinct reactoin whereas a plastic clay has no reaction. Inorganic silts, such as a typical rock flour, show a moderately quick reaction.

DRY STRENGTH (Crushing characteristics)

After removing particles larger than No. 40 sieve size, mold a pat of soil to the consistency of putty, adding water if necessary. Allow the pat to dry completely by oven, sun, or air drying, and then test its strength by breaking and crumbling between the fingers. This strength is a measure of the character and quality of the colloidal fraction contained in the soil. The dry strength increases with increasing plasticity.

High dry strength is characteristic for clays of the CH group. A typical inorganic silt possesses only very slight dry strength. Silty fine sand and silts have about the same slight dry strength, but can be distinguished by the feel when powdering the dried specimen. Fine sand feels gritty whereas a typical silt has the smooth feel of flour.

TOUGHNESS (Consistency near plastic limit)

After removing particles larger that the No. 40 sieve size, a specimen of soil about one-half inch cube in size is molded to the consistency of putty. If too dry, water must be added and if sticky, the specimen should be spread out in a thin layer and allowed to lose some moisture by evaporation. Then the specimen is rolled out by hand on a smooth surface of between the palms into a thread about one-eight inch in diameter . The thread is then folded and rerolled repeatedly. During this manipulation the moisture content is gradually reduced and the specimen stiffens, finally loses its plasticitiy, and crumbles when the plastic limit is reached.

After the thread crumbles, the pieces should be lumped together and a slight kneading action continued until the lump crumbles.

The tougher the thread near the plastic limit and the stiffer the lump when it finally crumbles, the more potent is the colloidal clay fraction in the soil. Weakness of the thread at the plastic limit and quick loss of coherence of the lump below the plastic limit indicate either inorganic clay of low plasticity, or materials such as kaolin-type clays and organic clays which occure below the A-line.

Highly organic clays have a very weak and spongy feel at the plastic limit.

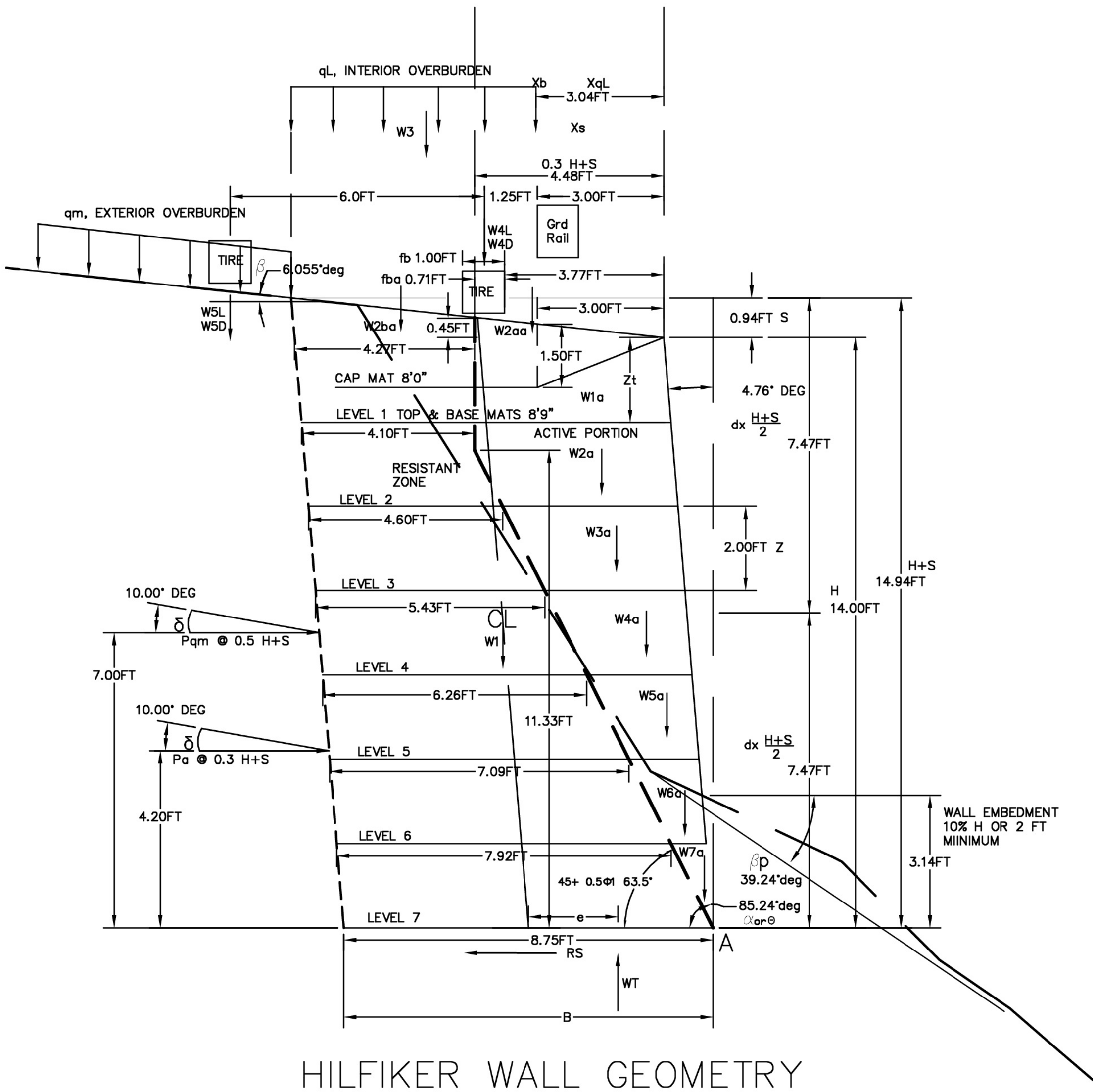
ADOPTED BY: CORPS OF ENGINEERS AND BUREAU OF RECLAMATION-JANUARY 1962

IO3-D-347

GRICE ENGINEERING INC
561A Brunken Avenue
Salinas, California 93901
Salinas: (831) 422-9619
Monterey: (831) 375-1198 FAX: (831) 422-1896

SOIL CLASSIFICATION CHART conforms to Unified Soils Classification and ASTM D2487

APPENDIX C



HILFIKER WALL GEOMETRY

Site Values

Horizontal seismic acceleration factor

$C_A := 1$

$A := 0.20 \cdot \frac{ft}{sec^2}$ $A_m := \left(1.45 \cdot \frac{ft}{sec^2} - A\right) A \cdot \left(\frac{sec^2}{ft}\right)$ $A_m = 0.25 \frac{ft}{sec^2}$

$K_h := 0.15$

Soil Values

Internal friction angle of reinforced wall, degrees $\Phi_1 := 37$

Cohesion of same $c_1 := .05 \cdot \frac{kip}{ft^2}$

Cohesion reduction factor $cr_1 := .10$

Friction angle for sliding at foundation soil, degrees $\Phi_2 := 42$

Cohesion of same $c_2 := .50 \cdot \frac{kip}{ft^2}$

Cohesion reduction factor $cr_2 := .10$

Friction angle of soil for retained active soil pressure, degrees $\Phi_3 := 33$

Cohesion of same $c_3 := .06 \cdot \frac{kip}{ft^2}$

Cohesion reduction factor $cr_3 := .10$

Angle of wall friction, degrees $\delta := 10$

Unit weight of wall backfill, kips per cubic foot $UW_1 := .120 \cdot \frac{kip}{ft^3}$

$\gamma_1 := UW_1$

Unit weight of Wall Overburden, kips per cubic foot $UW_2 := .115 \cdot \frac{kip}{ft^3}$

$\gamma_2 := UW_2$

Unit weight of Inplace Soils, kips per cubic foot $UW_3 := .115 \cdot \frac{kip}{ft^3}$

$\gamma_3 := UW_3$

WALL GEOMETRY

Slope of wall face to the horizontal, degrees	$\alpha := 85.24$
	$\theta := \alpha$
Slope of ground surface to the horizontal behind the wall, degrees	$\beta := 6.055$
Wall height, top mat to bottom mat, ft	$H := 14.00 \cdot ft$
Base Depth (mat depth), feet	$B := 8.75 \cdot ft$
Depth of top mat	$Z_t := 2 \cdot ft$
Fill over top mat, feet	$S := 0.94 \cdot ft$
Distance from face of wall to level backfill, feet	$X_s := 0.00 \cdot ft$
Distance from face of wall to full overburden, feet	$X_{qL} := 0.00 \cdot ft$
Width of wall considered in calculations	$W_w := 1 \cdot ft$

Constants for External and Internal Stability (From Hilfiker)

Constant C.1 at top of wall $C_{1t} := 0.65$

Constant C.1 at 20 feet $C_{120} := 1 - \mathrm{sind}(\Phi_1)$

Constant C.2 $C_2 := 36.5$

Constant C.3 $C_3 := 17.6$

Constant C.4 $C_4 := .55$

Constant C.5 $C_5 := 2.143 \cdot ft$

Tire Load, set values marked to 0 when no access is present

Height top of wall to bottom of tire, feet	$Y_b := 0.45 \cdot ft$
Tire Width, feet	$f_b := 1.0 \cdot ft$
Distance from face of wall to edge of tire, feet	$X_b := 3.77 \cdot ft$
Tire live load, kips per square foot	$W_{bL} := 1.00 \cdot \frac{kip}{ft^2}$
Tire dead load, kips per square foot	$W_{bd} := 2.5 \cdot \frac{kip}{ft^2}$

OVERBURDEN ABOVE REINFORCED PORTION OF WALL (above level S)

Overburden dead load expressed as equivalent feet of soil, feet	$q_{Ld} := 0 \cdot ft$
Overburden live load expressed as equivalent feet of soil, feet	$q_{LL} := 0 \cdot ft$

OVERBURDEN ABOVE GRADE REAR OF WALL (above level S)

Overburden dead load expressed as equivalent feet of soil, feet	$q_{md} := 0 \cdot ft$
Overburden live load expressed as equivalent feet of soil, feet	$qmL := 0ft$

Wall Type and Reinforcing

Hilfiker WWW

Reinforcing W7 x W7 - 8" x 21" WWF

Corrosion Treatment Hot dipped galavanizing, 2 o.z per square foot

Yield Strength of soil reinforcing wire, KSI $F_y := 65\frac{kip}{in^2}$

Vertical Spacing of soil reinforcing mats, feet $Z := 2 \cdot ft$

Vertical Spacing of top mat, feet $Z_t := 1.5 \cdot ft$

Corrosion Allowance, mils $C := 56 \cdot mil$

WIRE PARRALLEL TO FACE OF WALL

Wire diameter, before corrosion allowance, in $d_L := 0.299 \cdot in$

Wire diameter, after corrosion allowance, in $D_L := 0.299 \cdot in - C$

$D_L = 0.243\, in$

Cross sectional area after corrosion allowance, in^2 $A_L := \left(\frac{D_L}{2}\right)^2 \cdot \pi$ $A_L = 0.046\, in^2$

Spacing of Soil Reinforcing Wire, inches $Sw_L := 21 \cdot in$

Longitudinal wires per foot depth $W_{Lc} := \frac{1 \cdot ft}{Sw_L}$ $W_{Lc} = 0.571$

WIRE PERPENDICULAR TO FACE OF WALL

Wire diameter, before corrosion allowance, in $d_t := 0.299 \cdot in$

Wire diameter, after corrosion allowance, in $D_t := 0.299 \cdot in - C$

$t := D_t$ $D_t = 0.243\, in$

Cross sectional area after corrosion allowance, in^2 $A_t := \left(\frac{D_t}{2}\right)^2 \cdot \pi$ $A_t = 0.046\, in^2$

Transverse spacing of soil reinforcing wire, in. $Sw_t := 8 \cdot in$

$S_t := Sw_t$ Transverse wires per foot depth $W_{tc} := \frac{1 \cdot ft}{Sw_t}$ $W_{tc} = 1.5$

Effective strength of reinforcing per foot width $T_{La} := \frac{W_{tc} \cdot A_t \cdot F_y}{W_w}$ $T_{La} = 4.522 \frac{kip}{ft}$

Reinforcing ratio (ratio of width of soil reinforcing to width of wall) $R_F := 1.0$

EXTERNAL STABILITY

Active pressure coefficient of soil behind reinforced soil

$$K_a := \frac{\left(\text{sind}(\alpha + \Phi_3)\right)^2}{(\text{sind}(\alpha))^2 \cdot \text{sind}(\alpha - \delta) \cdot \left[1 + \sqrt{\left(\frac{\text{sind}(\Phi_3 + \delta) \cdot \text{sind}(\Phi_3 - \beta)}{\text{sind}(\alpha - \delta) \cdot \text{sind}(\alpha + \beta)}\right)}\right]^2} \qquad K_a = 0.33$$

Weight of wall mass, top mat to bottom mat	$W_1 := UW_1 \cdot H \cdot B$	$W_1 = 14.7 \frac{kip}{ft}$
Moment arm of wall mass about A	$h_1 := 0.5 \cdot B$	$h_1 = 4.375\,ft$
Weight of wedge fill above top mat	$W_{2a} := UW_1 \cdot S \cdot 0.5 \cdot X_s$	$W_{2a} = 0 \frac{kip}{ft}$
Moment arm of wedge fill above top mat about A	$h_{2a} := \frac{2 \cdot X_s}{3}$	$h_{2a} = 0\,ft$
Weight of rect.fill above top mat, level grade	$W_{2b} := UW_1 \cdot S \cdot (B - X_s)$	$W_{2b} = 0.987 \frac{kip}{ft}$
Moment arm of rect fill mass about A	$h_{2b} := X_s + \left(\frac{B - X_s}{2}\right)$	$h_{2b} = 4.375\,ft$
Interior overburden above top lift	$W_3 := UW_1 \cdot q_{Ld} \cdot (H - X_{qL})$	$W_3 = 0 \frac{kip}{ft}$
Moment arm of interior overburden about A	$h_3 := X_{qL} + \left[\frac{(B - X_{qL})}{2}\right]$	$h_3 = 4.375\,ft$
Tire load, Live	$W_{4L} := W_{bL} \cdot f_b$	$W_{4L} = 1 \frac{kip}{ft}$
Tire load, Dead	$W_{4D} := W_{bd} \cdot f_b$	$W_{4D} = 2.5 \frac{kip}{ft}$
Moment arm of tire load about A	$h_4 := \left(X_b + \frac{f_b}{2}\right)$	$h_4 = 4.27\,ft$

Active soil pressure behind wall $P_a := \frac{\left[(H+S)^2 \cdot K_a \cdot UW_2 \cdot \text{cosd}(\delta)\right]}{2}$ $P_a = 4.168 \frac{\text{kip}}{\text{ft}}$

Internal resistance due to cohesion $P_c := 2 \cdot c_3 \cdot cr_3 \cdot (H+S) \cdot \sqrt{K_a}$ $P_c = 0.103 \frac{\text{kip}}{\text{ft}}$

Total active pressure at back of wall $P_{at} := P_a - P_c$ $P_{at} = 4.065 \frac{\text{kip}}{\text{ft}}$

Point of application on back of wall $h_a := \frac{(H+S)}{3}$ $h_a = 4.98\ \text{ft}$

Lateral pressure from overburden back of wall $P_{qm} := q_{md} \cdot (H+S) \cdot K_a \cdot UW_1 \cdot \text{cosd}(\delta)$ $P_{qm} = 0 \frac{\text{kip}}{\text{ft}}$

lateral point of application $h_q := \frac{(H+S)}{2}$ $h_q = 7.47\ \text{ft}$

Passive Pressure

Angle of slope below toe $\beta_p := -39$ Per section developed by Whitson Eng.

Angle of foundation face from horizontal $\alpha = 85.24$

$$K_p := \left[\frac{(\mathrm{cosd}(\Phi_2))}{1-\sqrt{\left[(\mathrm{sind}(\Phi_2))\cdot\left[(\mathrm{sind}(\Phi_2)) + (\mathrm{cosd}(\Phi_2)\cdot\mathrm{tand}(\beta_p))\right]\right]}}\right]^2 \qquad K_p = 0.89$$

Embedment of wall below grade $D_e := 3.14\cdot\mathrm{ft}$

Embedment depth to ignore $b_{nc} := 0.5\cdot\mathrm{ft}$

Width of foundation to consider $E_b := 1$

$$P\phi_p := \left[\left[\frac{(\gamma_3)}{2}\right]\cdot(D_e - b_{nc})^2\cdot K_p\cdot(E_b)\right] \qquad P\phi_p = 0.357\,\frac{\mathrm{kip}}{\mathrm{ft}}$$

$$Pc_p := \left[2\cdot c_3\cdot cr_3\cdot\sqrt{K_p}\cdot(D_e - b_{nc})\cdot(E_b)\right] \qquad Pc_p = 0.03\,\frac{\mathrm{kip}}{\mathrm{ft}}$$

$$P_p := P\phi_p + Pc_p \qquad P_p = 0.387\,\frac{\mathrm{kip}}{\mathrm{ft}}$$

Passive moment resisting retained materials. Ph acts at approx. 1/3 up from bottom of foundation

$$Pf_m := (P\phi_p + Pc_p)\cdot\frac{(D_e - b_{nc})}{3} \qquad Pf_m = 340.177\,\mathrm{lbf} \qquad h_{pfm} := \frac{D_e}{3}$$

EXTERNAL STABILITY - STATIC

Moment Feet about A, (kip)

$MR_1 := W_1 \cdot h_1$ $MR_1 = 64.313\,\text{kip}$

$MR_{2a} := W_{2a} \cdot h_{2a}$ $MR_{2a} = 0\,\text{kip}$

$MR_{2b} := W_{2b} \cdot h_{2a}$ $MR_{2b} = 0\,\text{kip}$

$MR_3 := W_3 \cdot h_3$ $MR_3 = 0\,\text{kip}$

$MR_{4D} := W_{4D} \cdot h_4$ $MR_{4D} = 10.675\,\text{kip}$

$MR_{4L} := W_{4L} \cdot h_4$ $MR_{4L} = 4.27\,\text{kip}$

$W_t := \left(W_1 + W_{2a} + W_{2b} + W_3 + W_{4D} + W_{4L}\right)$ $W_t = 19.187\,\frac{\text{kip}}{\text{ft}}$

$MR_t := \left(MR_1 + MR_{2a} + MR_{2b} + MR_3 + MR_{4D} + MR_{4L}\right)$ $MR_t = 79.258\,\text{kip}$

Moment Feet, (kip)

$M_{OTa} := P_a \cdot h_a$ $M_{OTa} = 20.759\,\text{kip}$

$M_{OTq} := P_{qm} \cdot h_q$ $M_{OTq} = 0\,\text{kip}$

$P_t := P_{at} + P_{qm}$ $P_t = 4.065\,\frac{\text{kip}}{\text{ft}}$

$M_{OTT} := M_{OTa} + M_{OTq}$ $M_{OTT} = 20.759\,\text{kip}$

Dead Loads

$$FS_{dsliding} := \frac{\left[\left(W_1 + W_{2a} + W_{2b} + W_{4D}\right)\cdot \mathrm{tand}\left(\Phi_2\right) + \left(B\cdot c_2\cdot cr_2\right) + P_p\right]}{P_a + P_{qm}}$$

$FS_{dsliding} = 4.126$ to be >=1.5 OK

$$FS_{doverturning} := \frac{\left(MR_1 + MR_{2a} + MR_{2b} + MR_{4D} + Pf_m\right)}{M_{OTa}}$$

to be >=2.0 OK

$FS_{doverturning} = 3.629$

Foundation Pressure Excentricity

$$e_{d1} := \left(\frac{B}{2}\right) - \frac{\left(MR_1 + MR_{2a} + MR_{2b} + MR_{4D} - M_{OTa}\right)}{W_1 + W_{2a} + W_{2b} + W_{4D}}$$

<=B/4 for rock foundation

$e_{d1} = 1.393\ \mathrm{ft}$ <= $M4 := \frac{B}{4}$ $M4 = 2.188\ \mathrm{ft}$ OK

$$FP_{d1} := \frac{\left(W_1 + W_{2a} + W_{2b} + W_{4D}\right)}{B - 2\cdot e_{d1}}$$

<=Allowable Bearing Capacity

$FP_{d1} = 3.05\ \frac{kip}{ft^2}$ Per soils report = 4 kip/ft^2 OK

Live Loads

$$FS_{Lsliding} := \frac{\left[\left(W_1 + W_{2a} + W_{2b} + W_{4D} + W_{4L}\right)\cdot \mathrm{tand}\left(\Phi_2\right) + \left(B\cdot c_2\cdot cr_2\right) + P_p\right]}{P_a + P_{qm}}$$

$FS_{Lsliding} = 4.342$ >=1.5 OK

$$FS_{Loverturning} := \frac{\left(MR_1 + MR_{2a} + MR_{2b} + MR_{4D} + MR_{4L} + Pf_m\right)}{M_{OTT}}$$

$FS_{Loverturning} = 3.834$

>=2.0 OK

Footing Pressure Excentricity

$$e_{dl2} := \left(\frac{B}{2}\right) - \frac{\left(MR_t - M_{OTT}\right)}{W_t}$$

<=B/4 for rock foundation

$e_{dl2} = 1.326\,ft$ <= $M4 = 2.188\,ft$

$$FP_{dl2} := \frac{W_t}{B - 2\cdot e_{dl2}}$$

<=Allowable Bearing Capacity OK

$FP_{dl2} = 3.147\,\frac{kip}{ft^2}$

Per soils report = 4 kip/ft^2 + 1/3 = 5.3 kip/ft^2

SEISMIC LOADING

For Seismic the safety factor is reduced by 0.75

External Seismic Force

Seismic Loading of Retained Earth Per Day; 2002; EQ. 10.7

$K_h := (K_a)\cdot(A_m)\cdot\frac{sec^2}{ft}$ $K_h = 0.082$

$P_E := 0.5\cdot K_h\cdot\frac{\left[(H+S)^2\cdot\gamma_2\right]}{2}$ $P_E = 0.529\,\frac{kip}{ft}$

Applied at 2/3*H+S

$h_{ae} := \frac{(H+S)}{2}$ $h_{ae} = 7.47\,ft$

Internal Load Acceleration of mass within reinforced portion of wall including overburdens

Mass for inertial horizontal forces uses reduced width replacing B with 0.5*(H+S) as applicable

Weight of wall mass, top mat to bottom mat	$W_{1e} := UW_1\cdot H\cdot[0.5(H+S)]$	$W_{1e} = 12.55\,\frac{kip}{ft}$
Moment arm of inertial wall mass about A	$h_{1e} := 0.5\cdot H$	$h_{1e} = 7\,ft$
Weight of wedge fill above top mat	$W_{2ae} := UW_1\cdot S\cdot 0.5\cdot X_s$	$W_{2ae} = 0\,\frac{kip}{ft}$
Moment arm of wedge fill above top mat about A	$h_{2ae} := \left(H+\frac{S}{3}\right)$	$h_{2ae} = 14.313\,ft$
Weight of rect.fill above top mat, level grade	$W_{2be} := UW_1\cdot S\cdot\left[0.5(H+S)-X_s\right]$	$W_{2be} = 0.843\,\frac{kip}{ft}$
Moment arm of rect fill mass about A	$h_{2be} := \left(H+\frac{S}{2}\right)$	$h_{2be} = 14.47\,ft$
Interior overburden above top lift	$W_{3e} := UW_1\cdot q_{Ld}\cdot X_{qL}$	$W_{3e} = 0\,\frac{kip}{ft}$
Moment arm of interior overburden about A	$h_{3e} := (H+S)$	$h_{3e} = 14.94\,ft$
Tire load, Live	$W_{4Le} := W_{bL}\cdot f_b$	$W_{4Le} = 1\,\frac{kip}{ft}$
Tire load, Dead	$W_{4De} := W_{bd}\cdot f_b$	$W_{4De} = 2.5\,\frac{kip}{ft}$
Moment arm of tire load about A	$h_{4e} := (H+S)$	$h_{4e} = 14.94\,ft$

INTERNAL Seismic Horizontal Load kip X Moment Arm feet

Moment Feet, (kip)

$MO_{1e} := W_{1e} \cdot h_{1e} \cdot K_h$ $MO_{1e} = 7.243\,kip$

$MO_{2ae} := W_{2ae} \cdot h_{2ae} \cdot K_h$ $MO_{2ae} = 0\,kip$

$MO_{2be} := W_{2be} \cdot h_{2be} \cdot K_h$ $MO_{2be} = 1.005\,kip$

$MO_{3e} := W_{3e} \cdot h_{3e} \cdot K_h$ $MO_{3e} = 0\,kip$

$MO_{4De} := W_{4De} \cdot h_{4e} \cdot K_h$ $MO_{4De} = 3.08\,kip$

$MO_{4Le} := W_{4Le} \cdot h_{4e} \cdot K_h$ $MO_{4Le} = 1.232\,kip$

$W_{te} := \left(W_{1e} + W_{2ae} + W_{2be} + W_{3e} + W_{4De} + W_{4Le}\right) \cdot K_h$ $W_{te} = 1.393\,\frac{kip}{ft}$

$MO_{te} := \left(MO_{1e} + MO_{2ae} + MO_{2be} + MO_{3e} + MO_{4De} + MO_{4Le}\right)$ $MO_{te} = 12.56\,kip$

EXTERNAL Seismic horizontal Load kip X Moment Arm feet

Moment Feet, (kip)

$M_{OTae} := P_E \cdot h_{ae}$ $M_{OTae} = 3.952\,kip$

$P_{tE} := P_t + P_E + W_{te}$ $P_{tE} = 5.987\,\frac{kip}{ft}$

$M_{OTTE} := M_{OTT} + M_{OTae} + MO_{te}$ $M_{OTTE} = 37.271\,kip$

Sliding Resistance

$$FS_{Se} := \frac{\left[W_t \cdot tand\left(\Phi_2\right) + \left(B \cdot c_2 \cdot cr_2\right) + P_p\right]}{P_{tE}}$$

$FS_{Se} = 3.023$ > $FS_{SEmin} := 1.1 \cdot 0.75$ $FS_{SEmin} = 0.825$ OK

Overturning Resistance

$FS_{Oe} := \frac{MR_t + Pf_m}{M_{OTTE}}$ $FS_{Oe} = 2.136$ > $FS_{OEmin} := 2.0 \cdot 0.75$ $FS_{OEmin} = 1.5$ OK

Footing Pressure Excentricity $e_e := \left[\left(\frac{B}{2}\right) - \frac{\left(MR_t - M_{OTT}\right)}{W_t}\right]$ <=B/4 for rock foundation

$e_e = 1.326\,ft$ <= $M4 = 1.458\,ft$

$FP_e := \frac{W_t}{\left(B - 2 \cdot e_e\right)}$ $FP_e = 3.147\,\frac{kip}{ft^2}$ <= Allowable Bearing Capacity

Per soils report = 4 kip/ft^2 + 1/3 = 5.3 kip/ft^2

Internal Soil Load $fps := \frac{e}{B}$ $fps = 0.311\,\frac{1}{ft}$

Internal Stability

$F_1 := 20\cdot\left(\frac{t}{S_t}\right)$ $\quad C := 2 \quad F_1 = 0.607 \quad Z_p := 1 \quad \alpha_{sc} := 1 \quad R_c := 1$

z_i Depth to each reinforcing layer from top of reinforced wall

Number of layers excluding top mat $D_i := 7$

Proposed mat depths in resistant zone	Proposed mat depths in active zone
$Le_1 := 4.27\cdot ft$	$La_1 := B - Le_1$
$Le_2 := 4.10\cdot ft$	$La_2 := B - Le_2$
$Le_3 := 4.60\cdot ft$	$La_3 := B - Le_3$
$Le_4 := 5.43\cdot ft$	$La_4 := B - Le_4$
$Le_5 := 6.26\cdot ft$	$La_5 := B - Le_5$
$Le_6 := 7.92\cdot ft$	$La_6 := B - Le_6$
$Le_7 := 7.92\cdot ft$	$La_7 := B - Le_7$

Total length in resistant zone

$$Le_T := \left(Le_1 + Le_2 + Le_3 + Le_4 + Le_5 + Le_6 + Le_7\right)$$

Total length in resistant zone

$$La_T := \left(La_1 + La_2 + La_3 + La_4 + La_5 + La_6\right)$$

Following repeats for each reinforcement horizon beginning at the top layer - 1

First layer down

$z_i := 2.00 \cdot ft$ $\quad K_r := K_a \cdot \left(\frac{z_i}{ft} \cdot -0.065 + 2.5 \right)$ $\quad K_r = 0.782$

Overburden $\quad \sigma_2 := W_{2b} + W_{2a} + W_{4D}$ $\quad \sigma_2 = 3.487 \frac{kip}{ft}$

$\sigma_v := \gamma_1 \cdot z_i \cdot ft + \sigma_2$ $\quad \sigma_v = 3.727 \frac{kip}{ft}$

$\sigma_H := K_r \cdot \sigma_v$ $\quad \sigma_H = 2.913 \frac{kip}{ft}$

Maximum tension in reinforcing layer

$T_{max1} := \frac{(\sigma_H)}{R_c}$ $\quad T_{max1} = 2.913 \frac{kip}{ft}$

Max allowable tension per foot width of wall

$T_{La} = 4.522 \frac{kip}{ft}$ $\quad$ >= Tmax1 $\quad FS_T := \frac{T_{La}}{T_{max1}}$ $\quad FS_T = 1.552$

Required embedment length in resistant zone

$L_{e1} := \frac{(1.5 \cdot T_{max1} \cdot ft)}{C \cdot F_1 \cdot \sigma_2 \cdot R_c \cdot \alpha_{sc}}$ $\quad L_{e1} = 1.031\, ft$

Proposed embedment length in resistant zone $\quad Le_1 = 4.27\, ft$

Weight of Active Zone and dead overburden

Areas of active zone, measured values

$W2_{aa} := 1.07 \cdot ft^2$ $\quad W2_{ba} := 0 \cdot ft^2$ $\quad W1_a := 9.19 \cdot ft^2$

Previously calculated weights $\quad W4_d := f_b \cdot W_{bd}$ $\quad W4_d = 2.5 \frac{kip}{ft}$

Weight of the Active Zone

$AZ_{w1} := W2_{aa} \cdot \gamma_2 + W4_d + W2_{ba} \cdot \gamma_2 + W1_a \cdot \gamma_1$ $\quad AZ_{w1} = 3.726 \frac{kip}{ft}$

Internal Seismic Evaluation

Maximum acceleration of the active zone

$$P_I := AZ_{w1} \cdot A_m \cdot \frac{sec^2}{ft} \qquad P_I = 0.931 \frac{kip}{ft}$$

Sum of the proposed embedment lengths $Le_T := 38.24 \cdot ft$

Distribution of horizontal acceleration of active zone on reinforcement level in evaluation

$$Ts_{md1} := P_I \cdot \frac{L_{e1}}{Le_T} \qquad Ts_{md1} = 0.025 \frac{kip}{ft}$$

Total force on reinforcememt level in evaluation

$$Ts_{total1} := T_{max1} + Ts_{md1}$$

For Seismic the safety factor is reduced by 0.75

$$Ts_{SF} := \frac{T_{La}}{\frac{\left(Ts_{total1} \cdot 0.75\right)}{R_c}} \qquad Ts_{SF} = 2.052$$

Factor of Safety In Zone

Second layer down

$z_i := 2\cdot ft + 2.0\cdot ft$ $\qquad K_r := K_a\cdot\left(\frac{z_i}{ft}\cdot -0.065 + 2.5\right)$ $\qquad K_r = 0.739$

Overburden $\qquad \sigma_2 := W_{2b} + W_{2a} + W_{4D}$ $\qquad \sigma_2 = 3.487\frac{kip}{ft}$

$\sigma_v := \gamma_1\cdot z_i\cdot ft + \sigma_2$ $\qquad \sigma_v = 3.967\frac{kip}{ft}$

$\sigma_H := K_r\cdot\sigma_v$ $\qquad \sigma_H = 2.931\frac{kip}{ft}$

Maximum tension in reinforcing layer

$T_{max1} := \frac{(\sigma_H)}{R_c}$ $\qquad T_{max1} = 2.931\frac{kip}{ft}$

Max allowable tension per foot width of wall

$T_{La} = 4.522\frac{kip}{ft}$ $\qquad$ >= Tmax1 $\qquad FS_T := \frac{T_{La}}{T_{max1}}$ $\qquad FS_T = 1.543$

Required embedment length in resistant zone

$L_{e1} := \frac{(1.5\cdot T_{max1}\cdot ft)}{C\cdot F_1\cdot\sigma_2\cdot R_c\cdot\alpha_{sc}}$ $\qquad L_{e1} = 1.038\,ft$

Proposed embedment length in resistant zone $\qquad Le_2 = 4.1\,ft$

Weight of Active Zone and dead overburden

Areas of active zone, measured values

$W2_{aa} := 1.07\cdot ft^2$ $\qquad W2_{ba} := 0\cdot ft^2$ $\qquad W2_a := 1.56\cdot ft^2 + 8.97\cdot ft^2$

Previously calculated weights $\qquad W4_d := f_b\cdot W_{bd}$ $\qquad W4_d = 2.5\frac{kip}{ft}$

Weight of the Active Zone

$AZ_{w2} := W2_{aa}\cdot\gamma_2 + W4_d + W2_{ba}\cdot\gamma_2 + W2_a\cdot\gamma_1$ $\qquad AZ_{w2} = 3.887\frac{kip}{ft}$

Internal Seismic Evaluation

Maximum acceleration of the active zone

$P_I := AZ_{w2} \cdot A_m \cdot \frac{sec^2}{ft}$ $P_I = 0.972 \frac{kip}{ft}$

Sum of the proposed embedment lengths $Le_T := 38.24 \cdot ft$

Distribution of horizontal acceleration of active zone on reinforcement level in evaluation

$Ts_{md1} := P_I \cdot \frac{L_{e1}}{Le_T}$ $Ts_{md1} = 0.026 \frac{kip}{ft}$

Total force on reinforcememt level in evaluation

$Ts_{total1} := T_{max1} + Ts_{md1}$ For Seismic the safety factor is reduced by 0.75

$$Ts_{SF} := \frac{T_{La}}{\frac{\left(Ts_{total1} \cdot 0.75\right)}{R_c}}$$

$Ts_{SF} = 2.039$ Factor of Safety In Zone

Third layer down

$z_i := 4 \cdot ft + 2 \cdot ft$ $\quad K_r := K_a \cdot \left(\frac{z_i}{ft} \cdot -0.065 + 2.5 \right)$ $\quad K_r = 0.696$

Overburden $\quad \sigma_2 := W_{2b} + W_{2a} + W_{4D}$ $\quad \sigma_2 = 3.487 \frac{kip}{ft}$

$\sigma_v := \gamma_1 \cdot z_i \cdot ft + \sigma_2$ $\quad \sigma_v = 4.207 \frac{kip}{ft}$

$\sigma_H := K_r \cdot \sigma_v$ $\quad \sigma_H = 2.928 \frac{kip}{ft}$

Maximum tension in reinforcing layer

$T_{max1} := \frac{(\sigma_H)}{R_c}$ $\quad T_{max1} = 2.928 \frac{kip}{ft}$

Max allowable tension per foot width of wall

$T_{La} = 4.522 \frac{kip}{ft}$ >= Tmax1 $\quad FS_T := \frac{T_{La}}{T_{max1}}$ $\quad FS_T = 1.545$

Required embedment length in resistant zone

$L_{e1} := \frac{(1.5 \cdot T_{max1} \cdot ft)}{C \cdot F_1 \cdot \sigma_2 \cdot R_c \cdot \alpha_{sc}}$ $\quad L_{e1} = 1.037\,ft$

Proposed embedment length in resistant zone $\quad Le_3 = 4.6\,ft$

Weight of Active Zone and dead overburden

Areas of active zone, measured values

$W2_{aa} := 2.23 \cdot ft^2$ $\quad W2_{ba} := 2.58 \cdot ft^2$ $\quad W3_a := 1.56 \cdot ft^2 + 9.44 \cdot ft^2 + 7.47 \cdot ft^2$

Previously calculated weights $\quad W4_d := f_b \cdot W_{bd}$ $\quad W4_d = 2.5 \frac{kip}{ft}$

Weight of the Active Zone

$AZ_{w3} := W2_{aa} \cdot \gamma_2 + W4_d + W2_{ba} \cdot \gamma_2 + W3_a \cdot \gamma_1$ $\quad AZ_{w3} = 5.27 \frac{kip}{ft}$

Internal Seismic Evaluation

Maximum acceleration of the active zone

$$P_I := AZ_{w3} \cdot A_m \cdot \frac{\text{sec}^2}{\text{ft}} \qquad P_I = 1.317 \frac{\text{kip}}{\text{ft}}$$

Sum of the proposed embedment lengths $Le_T := 38.24 \cdot \text{ft}$

Distribution of horizontal acceleration of active zone on reinforcement level in evaluation

$$Ts_{md1} := P_I \cdot \frac{L_{e1}}{Le_T} \qquad Ts_{md1} = 0.036 \frac{\text{kip}}{\text{ft}}$$

Total force on reinforcememt level in evaluation

$$Ts_{total1} := T_{max1} + Ts_{md1}$$

For Seismic the safety factor is reduced by 0.75

$$Ts_{SF} := \frac{T_{La}}{\frac{\left(Ts_{total1} \cdot 0.75\right)}{R_c}} \qquad Ts_{SF} = 2.035$$

Factor of Safety In Zone

Fourth layer down

$z_i := 6 \cdot ft + 2 \cdot ft$ $\qquad K_r := K_a \cdot \left(\frac{z_i}{ft} \cdot -0.065 + 2.5 \right)$ $\qquad K_r = 0.653$

Overburden $\qquad \sigma_2 := W_{2b} + W_{2a} + W_{4D}$ $\qquad \sigma_2 = 3.487 \frac{kip}{ft}$

$\sigma_v := \gamma_1 \cdot z_i \cdot ft + \sigma_2$ $\qquad \sigma_v = 4.447 \frac{kip}{ft}$

$\sigma_H := K_r \cdot \sigma_v$ $\qquad \sigma_H = 2.904 \frac{kip}{ft}$

Maximum tension in reinforcing layer

$T_{max1} := \frac{(\sigma_H)}{R_c}$ $\qquad T_{max1} = 2.904 \frac{kip}{ft}$

Max allowable tension per foot width of wall

$T_{La} = 4.522 \frac{kip}{ft}$ $\qquad$ >= Tmax1 $\qquad FS_T := \frac{T_{La}}{T_{max1}}$ $\qquad FS_T = 1.557$

Required embedment length in resistant zone

$L_{e1} := \frac{(1.5 \cdot T_{max1} \cdot ft)}{C \cdot F_1 \cdot \sigma_2 \cdot R_c \cdot \alpha_{sc}}$ $\qquad L_{e1} = 1.028\,ft$

Proposed embedment length in resistant zone $\qquad Le_4 = 5.43\,ft$

Weight of Active Zone and dead overburden

Areas of active zone, measured values

$W2_{aa} := 1.07 \cdot ft^2$ $\qquad W2_{ba} := 0 \cdot ft^2$

$W4_a := 8.97 \cdot ft^2 + 8.97 \cdot ft^2 + 7.47 \cdot ft^2 + 5.81 \cdot ft^2$

Previously calculated weights $\qquad W4_d := f_b \cdot W_{bd}$ $\qquad W4_d = 2.5 \frac{kip}{ft}$

Weight of the Active Zone

$AZ_{w4} := W2_{aa} \cdot \gamma_2 + W4_d + W2_{ba} \cdot \gamma_2 + W4_a \cdot \gamma_1$ $\qquad AZ_{w4} = 6.369 \frac{kip}{ft}$

Internal Seismic Evaluation

Maximum acceleration of the active zone

$$P_I := AZ_{w4} \cdot A_m \cdot \frac{sec^2}{ft} \qquad P_I = 1.592 \frac{kip}{ft}$$

Sum of the proposed embedment lengths $Le_T := 38.24 \cdot ft$

Distribution of horizontal acceleration of active zone on reinforcement level in evaluation

$$Ts_{md1} := P_I \cdot \frac{L_{e1}}{Le_T} \qquad Ts_{md1} = 0.043 \frac{kip}{ft}$$

Total force on reinforcememt level in evaluation

$$Ts_{total1} := T_{max1} + Ts_{md1}$$

For Seismic the safety factor is reduced by 0.75

$$Ts_{SF} := \frac{T_{La}}{\frac{\left(Ts_{total1} \cdot 0.75\right)}{R_c}} \qquad Ts_{SF} = 2.046$$

Factor of Safety In Zone

Fifth layer down

$z_i := 8 \cdot ft + 2 \cdot ft$ $K_r := K_a \cdot \left(\frac{z_i}{ft} \cdot -0.065 + 2.5 \right)$ $K_r = 0.61$

Overburden $\sigma_2 := W_{2b} + W_{2a} + W_{4D}$ $\sigma_2 = 3.487 \frac{kip}{ft}$

$\sigma_v := \gamma_1 \cdot z_i \cdot ft + \sigma_2$ $\sigma_v = 4.687 \frac{kip}{ft}$

$\sigma_H := K_r \cdot \sigma_v$ $\sigma_H = 2.86 \frac{kip}{ft}$

Maximum tension in reinforcing layer

$T_{max1} := \frac{(\sigma_H)}{R_c}$ $T_{max1} = 2.86 \frac{kip}{ft}$

Max allowable tension per foot width of wall

$T_{La} = 4.522 \frac{kip}{ft}$ >= Tmax1 $FS_T := \frac{T_{La}}{T_{max1}}$ $FS_T = 1.581$

Required embedment length in resistant zone

$L_{e1} := \frac{(1.5 \cdot T_{max1} \cdot ft)}{C \cdot F_1 \cdot \sigma_2 \cdot R_c \cdot \alpha_{sc}}$ $L_{e1} = 1.012 ft$

Proposed embedment length in resistant zone $Le_5 = 6.26 ft$

Weight of Active Zone and dead overburden

Areas of active zone, measured values

$W2_{aa} := 1.07 \cdot ft^2$ $W2_{ba} := 0 \cdot ft^2$

$W5_a := 9.19 \cdot ft^2 + 8.97 \cdot ft^2 + 7.47 \cdot ft^2 + 5.81 \cdot ft^2 + 4.15 \cdot ft^2$

Previously calculated weights $W4_d := f_b \cdot W_{bd}$ $W4_d = 2.5 \frac{kip}{ft}$

Weight of the Active Zone

$AZ_{w5} := W2_{aa} \cdot \gamma_2 + W4_d + W2_{ba} \cdot \gamma_2 + W5_a \cdot \gamma_1$ $AZ_{w5} = 6.894 \frac{kip}{ft}$

Internal Seismic Evaluation

Maximum acceleration of the active zone

$P_I := AZ_{w5} \cdot A_m \cdot \frac{sec^2}{ft}$ $P_I = 1.723 \frac{kip}{ft}$

Sum of the proposed embedment lengths $Le_T := 38.24 \cdot ft$

Distribution of horizontal acceleration of active zone on reinforcement level in evaluation

$Ts_{md1} := P_I \cdot \frac{L_{e1}}{Le_T}$ $Ts_{md1} = 0.046 \frac{kip}{ft}$

Total force on reinforcememt level in evaluation

$Ts_{total1} := T_{max1} + Ts_{md1}$ For Seismic the safety factor is reduced by 0.75

$Ts_{SF} := \frac{T_{La}}{\frac{(Ts_{total1} \cdot 0.75)}{R_c}}$ $Ts_{SF} = 2.075$ Factor of Safety In Zone

Sixth layer down

$z_i := 10\cdot ft + 2\cdot ft$ $\quad K_r := K_a\cdot\left(\frac{z_i}{ft}\cdot -0.065 + 2.5\right)$ $\quad K_r = 0.567$

Overburden $\quad \sigma_2 := W_{2b} + W_{2a} + W_{4D}$ $\quad \sigma_2 = 3.487\frac{kip}{ft}$

$\sigma_v := \gamma_1\cdot z_i\cdot ft + \sigma_2$ $\quad \sigma_v = 4.927\frac{kip}{ft}$

$\sigma_H := K_r\cdot\sigma_v$ $\quad \sigma_H = 2.795\frac{kip}{ft}$

Maximum tension in reinforcing layer

$T_{max1} := \frac{(\sigma_H)}{R_c}$ $\quad T_{max1} = 2.795\frac{kip}{ft}$

Max allowable tension per foot width of wall

$T_{La} = 4.522\frac{kip}{ft}$ $\quad$ >= Tmax1 $\quad FS_T := \frac{T_{La}}{T_{max1}}$ $\quad FS_T = 1.618$

Required embedment length in resistant zone

$L_{e1} := \frac{(1.5\cdot T_{max1}\cdot ft)}{C\cdot F_1\cdot\sigma_2\cdot R_c\cdot\alpha_{sc}}$ $\quad L_{e1} = 0.99\,ft$

Proposed embedment length in resistant zone $\quad Le_6 = 7.92\,ft$

Weight of Active Zone and dead overburden

Areas of active zone, measured values

$W2_{aa} := 1.07\cdot ft^2$ $\quad W2_{ba} := 0\cdot ft^2$

$W6_a := 9.19\cdot ft^2 + 8.97\cdot ft^2 + 7.47\cdot ft^2 + 5.81\cdot ft^2 + 7.15\cdot ft^2 + 2.49\cdot ft^2$

Previously calculated weights $\quad W4_d := f_b\cdot W_{bd}$ $\quad W4_d = 2.5\frac{kip}{ft}$

Weight of the Active Zone

$AZ_{w6} := W2_{aa}\cdot\gamma_2 + W4_d + W2_{ba}\cdot\gamma_2 + W6_a\cdot\gamma_1$ $\quad AZ_{w6} = 7.553\frac{kip}{ft}$

Internal Seismic Evaluation

Maximum acceleration of the active zone

$P_I := AZ_{w6} \cdot A_m \cdot \frac{sec^2}{ft}$ $\qquad P_I = 1.888 \frac{kip}{ft}$

Sum of the proposed embedment lengths $\quad Le_T := 38.24 \cdot ft$

Distribution of horizontal acceleration of active zone on reinforcement level in evaluation

$Ts_{md1} := P_I \cdot \frac{L_{e1}}{Le_T}$ $\qquad Ts_{md1} = 0.049 \frac{kip}{ft}$

Total force on reinforcememt level in evaluation

$Ts_{total1} := T_{max1} + Ts_{md1}$ For Seismic the safety factor is reduced by 0.75

$Ts_{SF} := \frac{T_{La}}{\frac{\left(Ts_{total1} \cdot 0.75\right)}{R_c}}$ $\qquad Ts_{SF} = 2.12$ Factor of Safety In Zone

Seventh layer down

$z_i := 12 \cdot ft + 2 \cdot ft$ $\quad K_r := K_a \cdot \left(\frac{z_i}{ft} \cdot -0.065 + 2.5 \right)$ $\quad K_r = 0.524$

Overburden $\quad \sigma_2 := W_{2b} + W_{2a} + W_{4D}$ $\quad \sigma_2 = 3.487 \frac{kip}{ft}$

$\sigma_v := \gamma_1 \cdot z_i \cdot ft + \sigma_2$ $\quad \sigma_v = 5.167 \frac{kip}{ft}$

$\sigma_H := K_r \cdot \sigma_v$ $\quad \sigma_H = 2.71 \frac{kip}{ft}$

Maximum tension in reinforcing layer

$T_{max1} := \frac{(\sigma_H)}{R_c}$ $\quad T_{max1} = 2.71 \frac{kip}{ft}$

Max allowable tension per foot width of wall

$T_{La} = 4.522 \frac{kip}{ft}$ >= Tmax1 $\quad FS_T := \frac{T_{La}}{T_{max1}}$ $\quad FS_T = 1.669$

Required embedment length in resistant zone

$L_{e1} := \frac{(1.5 \cdot T_{max1} \cdot ft)}{C \cdot F_1 \cdot \sigma_2 \cdot R_c \cdot \alpha_{sc}}$ $\quad L_{e1} = 0.959\,ft$

Proposed embedment length in resistant zone $\quad Le_6 = 7.92\,ft$

Weight of Active Zone and dead overburden

Areas of active zone, measured values

$W2_{aa} := 1.07 \cdot ft^2$ $\quad W2_{ba} := 0 \cdot ft^2$

$W7_a := 9.19 \cdot ft^2 + 8.97 \cdot ft^2 + 7.47 \cdot ft^2 + 5.81 \cdot ft^2 + 7.15 \cdot ft^2 + 2.49 \cdot ft^2 + 0.83 \cdot ft^2$

Previously calculated weights $\quad W4_d := f_b \cdot W_{bd}$ $\quad W4_d = 2.5 \frac{kip}{ft}$

Weight of the Active Zone

$AZ_{w7} := W2_{aa} \cdot \gamma_2 + W4_d + W2_{ba} \cdot \gamma_2 + W7_a \cdot \gamma_1$ $\quad AZ_{w7} = 7.652 \frac{kip}{ft}$

Internal Seismic Evaluation

Maximum acceleration of the active zone

$$P_I := AZ_{w7} \cdot A_m \cdot \frac{sec^2}{ft} \qquad P_I = 1.913 \frac{kip}{ft}$$

Sum of the proposed embedment lengths $Le_T := 38.24 \cdot ft$

Distribution of horizontal acceleration of active zone on reinforcement level in evaluation

$$Ts_{md1} := P_I \cdot \frac{L_{e1}}{Le_T} \qquad Ts_{md1} = 0.048 \frac{kip}{ft}$$

Total force on reinforcememt level in evaluation

$$Ts_{total1} := T_{max1} + Ts_{md1}$$

For Seismic the safety factor is reduced by 0.75

$$Ts_{SF} := \frac{T_{La}}{\frac{(Ts_{total1} \cdot 0.75)}{R_c}} \qquad Ts_{SF} = 2.186$$

Factor of Safety In Zone

REFERENCES

1. American Society of Civil Engineers, **Minimum Design Loads for Buildings and Other Structures, ASCE Standard ASCE/SEI 7-05 Including Supplement No. 1**, 2006, 385 pp.

2. Allen, C. R., 1975; **Geological criteria for evaluating seismicity,** GSA Bull. v. 86, p. 1041-1057.

3. Bailey, E. H., Irwim, W. P. and Jones, D. L., 1964, **Franciscan and Related Rocks, and their significance in the Geology of Western California,** CDMG Bulletin 183, 177 pp.

4. Bailey, E.H., Ed., 1966, **Geology of Northern California,** CDMG Bulletin 190, 507 pp.

5. Blair, M.L. and Spangle, W. E., 1979, **Seismic Safety and Land-Use Planning - Selected Examples from California,** USGS Professional Paper 941-B.

6. Bolt, B. A., 1975; **Geological Hazards,** Springer-Verlag, 328 p.

7. Bryant, W. A., 1985; **Faults in the Southern Monterey Bay area,** CDMG Fault Evaluation Report FER-167, 13 pp.

8. Bullis, K.C., 1980, **Environmental Constraints Analysis of Monterey County, Part I: Seismic and Geologic Hazards,** Monterey County Planning Department, General Update Program, Second printing June 1982, 54pp and appendices.

9. Bullis, K.C., 1981, **Environmental Constraints Analysis of Monterey County, Part I: Flood, Fire and Miscellaneous Hazards; Emergency Preparedness,** Monterey County Planning Department, General Update Program, pp 55-104 and appendices.

10. Burkland and Assoc., 1975, **Seismic Safety Element of the Monterey County General Plan,** 50 pp w/appendices.

11. Burkland and Associates, 1975; **Geotechnical study for the seismic safety element,** Monterey County, California, File No. K3-0113-M1, 125 pp.

12. California Department of Conservation, 1998, **Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada,** International Conference of Building Officials, Introduction & Maps.

13. 2016 California Building Code, **California Code of Regulations, Title 24, 2 Volumes, California Building Standards Commission**, Based on the 2015 International Building Code.

14. Clark, J. C. and Reitman, J. D., 1973. **Oligocene stratigraphy, tectonics, and paleogeography southwest of the San Andreas fault, Santa Cruz Mountains and Gabilan Range, California Coast Ranges**: U.S. G. S. Professional Paper 783, 18 p.

15. Clark, J. C., Diblee, T. W. Jr., Greene, H. G., and Bowen, O. E., Jr., 1974, **Preliminary geologic map of the Monterey and Seaside 7.5 minute quadrangles, Monterey County, California, with emphasis on active faults,** USGS Miscellaneous Field Studies Map MF-577.

16. Clark, Joseph C., Dupré, William R., & Rosenberg, Lewis I., **Geological Map of the Monterey and Seaside 7.5 minute Quadrangles, Monterey County, California: A Digital Database**, 1997, U. S. Department of the Interior, U. S. Geological Survey, Open-File Report 97-30, Map and Pamphlet, 26 pp.

17. Clark, Joseph C., Brabb, Earl E., & Rosenberg, Lewis I., 2000, **Geologic Map of the Spreckels 7.5-Minute Quadrangle, Monterey County, California,** USGS/Department of the Interior, Map MF-2349 & Pamphlet, 22 pp.

18. Clark, Joseph C. & Rosenberg, Lewis I., March 1999, **Southern San Gregorio Fault Displacement: Stepover Segmentation VS. Through-Going Tectonics,** USGS /Department of the Interior-National Earthquake Hazards Reduction Program, Award number 1434-HQ-98-GR-00007, 22 pp without Appendices

19. Cleveland, G.B., 1975, **Landsliding in Marine Terrace Terrain, California,** CDMG Special Report 119, 24pp.

20. Compton, R. R., 1966; **Granitic and metamorphic rocks of the Salinian Block, California Coast Ranges,** CDMG Bulletin 190, p. 277-287.

21. Diblee, T. W. Jr., 1966; **Evidence for cumulative offset on the San Andreas fault in central and northern California,** CDMG Bulletin 190.

22. Diblee, T. W., Jr., 1999; **Geologic Map of the Monterey Peninsula and Vicinity, Monterey, Salinas, Point Sur, and Jamesburg 15-Minute Quadrangles, Monterey County, California,** Diblee Geological Foundation Map #DF-71.

23. Dickinson, William R., Duccea, Mihai, Rosenberg, Lewis I., Greene, H. Gary, Graham, Stephan A., Clark, Joseph C., Weber , Gerald E., Kidder, Steven, Ernst, W. Gary and Brabb, Earl E., 2005; **Net dextral slip, Neogene San Gregorio-Hosgri fault zone, coastal California: Geologic evidence and tectonic implications**, Geological Society of America, Special Paper 391, 43 pp.

24. Dittmer, E. and Stein, C., 1977, **Salinas Seismic Hazards Technical Report,** Department of Community Development, City of Salinas, 73 pp.

25. Dupre, W. R. and Tinsley, J. C. III, 1980, **Geology and liquefaction potential of northern Monterey and southern Santa Cruz, California**: USGS Miscellaneous Field Studies Map 1199, Scale 1:62,500, 2 sheets.

26. Dupre, W. R., 1990, **Maps Showing Geology and Liquefaction Susceptibility of Quaternary Deposits in the Monterey, Seaside, Spreckels and Carmel Valley Quadrangles, Monterey County, CA**, U. S. Geological Survey and University of Huston, Map #MF-2096, 2 Sheets

27. Durham, D.L., 1974; **Geology of the Southern Salinas Valley Area, California**, USGS Professional Paper 819, 111 pp.

28. Greene, H. G., Lee, W.H.K., McCulloch, D.S., and Brabb, E.E., 1973; **Faults and Earthquakes in the Monterey Bay Region, California**, USGS MF 518, maps and paper, 14pp.

29. Greene, H. G., 1977; **Geology of the Monterey Bay region,** USGS Open-File Report p. 77-718.

30. Hays, W.W., 1980, **Procedures for Estimating Earthquake Ground Motions,** USGS Professional Paper 1114, 77 pp.

31. Jennings, C. W., and Strand, R. G., 1958; **Geologic Map of California, Olaf P. Jenkins edition, Santa Cruz sheet,** Scale 1:250,000, third printing 1971.

32. Jennings, C. W., and Bryant, W. A., 2010 **Fault Activity Map of California: California Geological Survey Geologic Data Map**, No. 6, Map Scale 1:750,000, Includes "An Explanatory Text to Accompany the Fault Activity Map of California", 94 pp

33. Lee, L. Don, Gudson, Seldon and Kauffman, Marvin E., 1978, **Physical Geology,** 5th Ed., Prentice Hall, Inc, Englewwod Cliffs, New Jersey 07632, 490 pp.

34. Lindh, A. G., 1983; **Preliminary assessment of long-term probabilities for large earthquakes along selected fault segments of the San Andreas fault system in California,** USGS Open File Report 83-63, 15 p.

35. Nason, R. D., and Rogers, T. H., 1967; **Self-guiding map to active faulting in the San Juan Bautista quadrangle, conference on geologic problems of the San Andreas fault system,** Stanford University, scale 1:24,000.

36. Nilsen, T.H., Diblee, T.W. Jr., and Blake, M.C. Jr., 1990, **Geology of the Central Diablo Range, CA,** Field Trip June 2-3.

37. Oakeshott, G. B., 1966; **San Andreas fault in the California Coast Range Province,** in Bailey, E. H., ed., **Geology of Northern California,** CDMG Bulletin 190, p. 357-373.

38. Plafker, G. and Galloway, J.P., eds., 1989 (approved for publication), **Lessons Learned from the Loma Prieta, California, Earthquake of October 17, 1989,** USGS Circular 1045, 48 pp.

39. Ray, R.G., 1960, **Aerial Photographs in Geologic Interpretation and Mapping,** USGS Professional Paper 373, seventh printing, 1984, 230 pp.

40. Real Estate Data Inc., 1980; **Aerial/Map Volume of Monterey County, California,** Photo 110, 2398 NW 119th St., Miami, FLA 33167, fifteenth edition.

41. Robbins, S.L., 1982, **Complete Bouguer Gravity, Aeromagnetic, and Generalized Geologic map of the Hollister 15-minute Quadrangle, CA,** Geophysical Investigations Map GP 945, 2 sheets, Scale 1:62,500.

42. Sarna-Wojcicki, A.M., Pampeyan, E.H. and Hall, N.T., 1975, **Maps Showing Recently Active Breaks Along the San Andreas Fault Between the Central Santa Cruz Mountains and the Northern Gabilan Range, CA,** 2 maps, text is on map 2, Scale 1:24,000.

43. Spangle, Wm. and Associates, Burkland and Associates, and Thorup, Richard R., July 1974; **Faults, Seismicity and Tsunami Hazards: Monterey County, California**: Part of Geological Report, County Map 3, File No. K4-0113-M1.

44. Tinsley, J. C. III, 1975, **Quaternary geology of northern Salinas Valley, Monterey County, California**: Stanford University PhD. thesis, 194 p., map, scale 1:62,500.

45. US Department of Agriculture, Soil Conservation Service, 1978, **Soil Survey, Monterey County, CA,** 226 pages and maps.

46. US Geological Survey, California Geological Survey, 2006**, Quaternary Fault and Fold Database for the United States,** access date same as report, from URL: http://earthquakes.usgs.gov/regional/qfaults/

47. US Geological Survey / California Geological Survey, 2006**, The USGS Store, Map Locator,** access date same as report, from URL: http://store.usgs.gov/ click on ‘map locator’

48. USGS Earthquake Hazards Program, **Seismic Design Values for Buildings-Earthquake Ground Motion Parameter**, URL: http://earthquake.usgs.gov/research/hazmaps/design/index.php

49. USGS Open File Report 88-398, 1988, **Probabilities of Large Earthquakes Occurring in California on the San Andreas Fault,** by the Working Group on California Earthquake Probabilities, 62 pp.

50. Wallace, R. E., 1970; **Earthquake recurrence intervals on the San Andreas fault,** GSA Bulletin, v. 81.

51. Wagner, David L., Greene, H. Gary, Saucedo, George J. and Pridmore, Cynthia L. Compiled by., Watkins, Sarah E., Little, Jason D. and Bizzarro, Joseph J. Digitalized by. 2002, California Department of Conservation, **Geologic Map of the Monterey 30' x 60' Quadrangle and Adjacent Areas, CA** , 3 maps and CD-ROM

52. Ward, P.L. and Page, R.A., 1989, **The Loma Prieta Earthquake of Oct 17, 1989,** USGS Pamphlet, Hdgen, L.D. and Troll, J.A., eds., second printing, revised, January 1990.

53. Wyss, M., 1979; **Estimating maximum expectable magnitude of earthquakes from fault dimensions,** Geology, v. 7, n. 7, p. 336-340.

54. Youd, T. L., and Hoose, S. N., 1978; **Historic ground failures in northern California triggered by earthquakes,** USGS Professional Paper P-993, p. 177