

GEOTECHNICAL INVESTIGATION

DYM RESIDENCE
7720 NORTH HIGHWAY 1
LITTLE RIVER, CALIFORNIA

Project Number 12934.01

May 4, 2020

Brunsing Associates, Inc.



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1.0 INTRODUCTION

This report presents the results of the geotechnical investigation performed by Brunsing Associates Inc. (BAI) for the planned Dym residence at 7720 North Highway 1, Little River, Mendocino County, California. The property, A. P. No. 121-280-15, is located approximately 1500 feet southeast of the mouth of the Little River at Van Damme Beach, as shown on the Vicinity Map, Plate 1. The Dym residence location is approximately two and one-half miles south-southeast of the town of Mendocino.

The purpose of our geotechnical investigation was to evaluate the site soil, bedrock and ocean bluff conditions in order to provide a geologic hazard assessment including bluff slope stability, bluff retreat rate, bluff setback and the potential effects of sea level rise. Our investigation also includes recommendations for site grading, support of concrete slabs-on-grade, structure foundation support and retaining wall design criteria. Our approach to providing the geotechnical guidelines for the design of the project utilized our knowledge of the soil, bedrock and geologic conditions in the site vicinity and experience with similar projects.

As outlined in our Professional Services Agreement, dated November 1, 2018, our scope of services for this geotechnical investigation included review of our previous reports on nearby properties, review of published geologic maps, study of aerial photographs, geologic reconnaissance and engineering and geologic analyses, in order to provide conclusions and recommendations regarding:

- Site preparation, grading and excavation characteristics;
- Suitable foundation type with design criteria and estimated settlement behavior;
- Seismic design criteria;
- Liquefaction analysis;
- Support of concrete slabs-on-grade, as appropriate;
- Bluff slope stability;
- Bluff setback;
- Site drainage;
- Retaining wall design criteria;
- Additional geotechnical services as appropriate.

2.0 INVESTIGATION AND LABORATORY TESTING

2.1 Research

As part of our investigation, we reviewed published geotechnical literature, including geologic, fault, and seismic hazard maps for the site and vicinity. We also reviewed previous geotechnical and geologic reports prepared by BAI on nearby properties. A list of selected published references reviewed for this investigation is presented in Appendix A.

2.2 Aerial Photograph Studies

Black and white, vertical aerial photographs that we reviewed for this study are dated 1942, June 30, 1963 and August 23, 1981. Other vertical aerial photographs that we examined were



obtained from Google Earth and the California Coastal Records Project (www.californiacoastline.org). The photographs are dated April 18, 1986, June 13, 1993, September 10, 1998, June 3, 2003, June 30, 2004, December 31, 2004, June 11, 2005, April 27, 2006, June 25, 2006, May 24, 2009, April 24, 2010, May 31, 2012, August 17, 2013 and July 2, 2018. The 1998 photo is black and white, all others are in color.

In addition to reviewing vertical aerial photographs, we also obtained oblique-angle aerial photographs from the California Coastal Records Project (color) dated 1972, October 5, 1979, June 1987, November 14, 2002, October 4, 2005, September 27, 2009, September 27, 2013 and October 4, 2019. We qualitatively compared the aerial photographs to look for changes in the property that may be due to erosion. The 1979 and 2019 coastline oblique aerial photographs are presented on Plate 2.

2.3 Field Reconnaissance

On April 23, 2019, our principal geotechnical engineer performed a site reconnaissance, taking photographs of the bluff and the area for the planned residence. He also staked the future boring locations and marked for underground service alert.

On May 13, 2019, our project engineer and staff geologist kayaked to the site from Van Damme Beach and took several photographs of the bluff face from various vantage points. Our BAI staff members additionally landed on the beaches directly below and adjacent to the subject site to photograph and investigate the bluff face for any apparent faults, landslides, bedding, joints, and other evidence for potential bluff instability. Our principal engineering geologist observed the site during the field operations.

2.4 Subsurface Exploration

Subsurface exploration was performed at the site on May 13, 2019. The exploration consisted of logging three vertical test borings, B-1, B-2 and B-3 that were drilled to depths of 15.5, 14.5, and 17.5 feet below ground surface (bgs) respectively. The borings were drilled using a track-mounted drill rig which utilized a four-inch diameter solid stem auger. Boring locations are shown on the Site Map, Plate 3.

Our staff geologist and principal engineering geologist made a descriptive log of each test boring and obtained relatively undisturbed tube samples of the site soil and bedrock materials encountered for visual classification and laboratory testing. The tube samples were obtained using a 3.0-inch outside diameter modified California (CA) split-barrel sampler and a 2.5-inch outside diameter modified California (CM) split-barrel sampler. The samplers were driven by a 140 pound drop hammer falling 30 inches per blow. Blow counts required to drive the CA and CM samplers were converted to Standard Penetration Test (SPT) blow counts for correlation with empirical test data using conversion factors of 0.64 and 0.79 respectively. Blow counts are presented on the boring logs.

The test borings showing the various soil and bedrock materials encountered and the depth at which the samples were obtained, are presented on Plates 4, 5 and 6. The soils are classified in



accordance with the Unified Soil Classification System outlined on Plate 7. The soil and bedrock descriptive properties are presented on Plate 8 and 9, respectively.

2.5 Laboratory Testing

The soil and bedrock samples obtained during our subsurface exploration were transported to our laboratory and examined to confirm field classification. Laboratory tests were performed on selected samples to estimate their pertinent geotechnical engineering characteristics. Laboratory testing consisted of moisture content, dry density and unconsolidated-undrained triaxial compression tests.

The test results are presented opposite each sample tested on the boring logs. A key to test data is provided on Plate 6. In addition, triaxial compression tests data results are presented on Plate 10.

3.0 SITE CONDITIONS

The property is on an ocean bluff in the southwesterly portion of the community of Little River. The property, back of the ocean bluffs, consist of a gently sloping, about 10 horizontal to one vertical (10H:1V), terrace. The planned building site spans a small, grass covered, sea stack that projects up the terrace, as shown in Site Photograph A, Plate 11. The bluffs are approximately 40 feet in height with slope gradients of approximately 0.5H:1V, with areas that are near vertical.

A small southwest-trending peninsula juts out into the ocean at the west end of the property. The peninsula is part of a ridge, a linear series of rocks that are mostly under water, except at minus tides, that form the southerly boundary of Van Damme Cove. North of the peninsula is Van Damme Beach. Southwest of the peninsula, there are two small, pocket beaches. The larger of the two beaches is directly below the bluff nearest the planned building site. At the time of our reconnaissance, a large bleached-wood log was lying across the approximate middle of the larger beach (Site Photographs B and C, Plate 12).

Site vegetation consists of grass and weeds across the terrace with stands of mostly pine trees in the northwest corner of the property and in the approximately center of the property, east of the proposed building site. The bluffs are mostly bare bedrock with areas of weeds and brush (Site Photograph D, Plate 13). One area of the bluffs contains a dense mass of Pampas Grass (Site Photograph E, Plate 14).

No surface water was observed in the vicinity of the planned building site during our investigation. No groundwater was encountered in our May 2019 test borings. One area of iron-mineralized groundwater seepage on the lower bluffs was observed, as shown on Site Photograph E, Plate 14.

4.0 SITE GEOLOGY AND SOILS

Site bedrock consists of Cretaceous-Tertiary Period Coastal Belt Franciscan Complex sedimentary bedrock consisting of sandstone and shale. The sandstone exposed in the lower portions of the bluff on the property is gray to dark gray, intensely to little fractured, friable to



hard, and little to deeply weathered. Silty sandstone and shale encountered in our borings around the planned building site, are orange-brown and gray, crushed to intensely fractured, friable to moderately hard, and moderately to deeply weathered. From surficial observations, the sea stack at the building site is comprised of moderately hard sandstone. Practical drilling refusal was encountered in moderately hard sandstone at 14.5 feet in boring B-2. Definitive bedding orientations were lacking in the Franciscan Complex bedrock from field observations. Open File Report 83-15 shows bedding orientations in the site vicinity consisting of a northwest trending strike with moderate dips, 43 to 63 degrees from horizontal, to the northeast.

Pleistocene Epoch terrace deposits overlie the Franciscan bedrock at the site. These shallow marine and beach deposits encountered in our borings vary from approximately one foot in thickness near the sea stack to about 12 feet in thickness towards the bluff. The upper terrace deposits are comprised of approximately one foot to three feet of variable dark brown silty sand and sandy silt topsoils that are loose or soft, porous and contain roots. Occasional rock fragments are interspersed in these loose or soft topsoils. Below the topsoils, the terrace deposits consist of light to dark brown and orange-brown, gravelly silty sands, sandy silts, gravelly clays, and sandy gravels that are loose or soft to medium dense or medium stiff. The terrace deposits appear low in expansion potential (tendency for volume change with changes in moisture content).

The beach deposits at the bluff toes are comprised of sands, gravels and some cobbles. The beach deposits vary in thickness seasonally.

There is evidence of recent bluff erosion, likely a small landslide, located at the bluff edge west-southwest of the planned building area. From the vantage point on the lower beach, BAI staff members observed landslide debris consisting of rock fragments, terrace deposit sediments, and disturbed vegetation (pampas grass) including tree remains, overlying the present-day beach.

The subject site has no known or visible active faults trending through or in the vicinity of the planned building area. The underlying Franciscan Complex is visibly folded, faulted, and jointed throughout. However, the faults and joints do not protrude through the overlying Pleistocene terrace deposits, which indicate that these features are not from recent seismic or tectonic activity. No faults are shown on or trending toward the site on the published maps we reviewed for the project.

The subject property is within the Coast Ranges geomorphic province, a zone of high seismic activity associated with the active San Andreas Fault system, which passes through the Mendocino County coastal area approximately three miles southwest of the site. The project site will be subject to strong ground shaking due to future, nearby earthquakes on this fault system during the lifetime of the proposed structure. In general, the intensity of ground shaking at the site will depend on the distance to the causative earthquake epicenter, the magnitude of the shock, and the response characteristics of the underlying earth materials.



5.0 DISCUSSIONS AND CONCLUSIONS

5.1 General

Based on the results of our reconnaissance and analyses, we conclude that the site is geotechnically suitable for the proposed residential development. The main geological and geotechnical considerations affecting the proposed construction are loose and porous near-surface soils, differential soil and bedrock conditions, potential settlement, bluff stability, bluff erosion and retreat rate, strong seismic shaking from future earthquakes and potential for liquefaction. These considerations and their possible mitigation measures are discussed below.

5.2 Settlement

The building site is mantled with weak topsoil and terrace deposits, underlain by supporting bedrock. The sea stack itself is comprised of hard bedrock covered by shallow soils. The topsoil consist of silty sands that are loose, porous, and moderately compressible. These soils could undergo erratic and detrimental settlement under the planned structure foundation loads. The loose to medium dense sandy terrace deposits below the topsoil may also be susceptible to detrimental settlement. The terrace deposits may contain clean sand zones that could be subject to densification during moderate or strong earthquake shaking. The topsoil and terrace deposits are not considered suitable for support of the planned structure foundation loads.

Utilizing a foundation system of drilled reinforced-concrete piers with interconnecting grade beams will allow the planned residence to gain uniform support within the stronger weathered bedrock underlying the terrace sands, mitigating the detrimental effects of differential settlement. Alternatively, a compacted fill pad could be constructed for support of footing foundations. Assuming foundations are designed and constructed in accordance with our recommendations, we estimate that the post-construction settlement due to foundation loads will be less than ½-inch. Detailed foundation recommendations are presented in Section 6.3 of this report.

5.3 Excavation Characteristics

Practical drilling refusal was encountered in boring B-2 at 14.5 feet. The sea stack appears to be comprised of moderately hard sandstone that will be difficult to excavate. A large excavator with a hoe ram may be needed to excavate into the sea stack, if needed.

5.4 Bluff Retreat

For our analysis, we used qualitative comparisons of the 1942 through 2018 vertical aerial photographs as well as the 1972 through 2019 oblique aerial photographs. Our qualitative comparison of the vertical and oblique aerial photographs shows minor changes to the bluff edges at the site, due to erosion and rock falls. Our site reconnaissance and quantitative review of aerial photographs indicate an average bluff retreat (erosion) rate along the ocean bluffs of approximately one inch per year.

BAI's estimated erosion rate is significantly less than the rate given in Open File Report 2007-1133 (approximately 16 inches per year) for this region. If the USGS rate were accurate, the



bluff edge would have retreated over 101 feet in the last 76 years (1942 to 2018, our earliest aerial photograph up to the most current). One hundred one feet of bluff loss would be easily visible in the aerial photographs that we reviewed for this project, which clearly is not the case.

5.5 Sea Level Rise Effects on Bluff Retreat

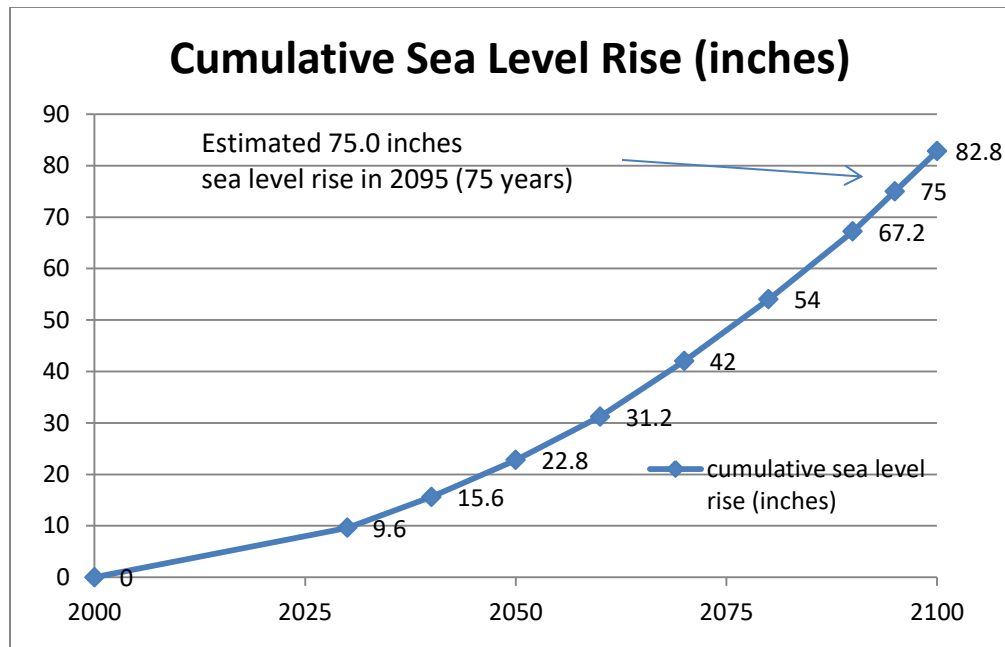
Rapid sea-level rise of approximately 400 to 450 feet occurred between 18,000 and 8,000 years before present, according to “Rising Seas in California”, Griggs, et al, 2017. Sea levels have remained relatively constant since that time. However, sea levels have started rising again. The California Coastal Commission (CCC) recently adopted the Science Update, dated November 7, 2018 to the 2015 Interpretive Guidelines for addressing Sea Level Rise in Local Coastal Programs and Coastal Development permits. The Science Update provides sea-level rise projections for the San Francisco coastal area, as follows in Table 1:

Table 1: Sea Level Rise Projections (Medium-High Risk Aversion)		
Time Period	Sea Level Rise (Feet)	Inches
2030	0.8	9.6
2040	1.3	15.6
2050	1.9	22.8
2060	2.6	31.2
2070	3.5	42.0
2080	4.5	54.0
2090	5.6	67.2
2100	6.9	82.8

The CCC requires a 75-year lifespan for new, coastal house construction or major remodel. As of 2020, we should have experienced a sea level rise of 5 inches, although we are not aware of any studies documenting such a rise. According to Dr. Mark Johnsson, former California Coastal Commission staff geologist, sea level rise on the west coast has been “lagging”, but will eventually come up to expectations. Recent projections show that by 2095, the sea level will be as much as 75 inches higher than present (2020).

Using the CCC’s economic lifespan of a building of 75 years, we must consider the effects of sea level rise for a structure built circa 2020 through 2095. For this discussion, we will assume a linear rate of sea level rise (which may or may not be the case) in order to estimate a projected sea level rise of approximately 75 inches (6.5 feet) by 2095.





Based upon historic aerial photographs and site observations, the current historic, average bluff retreat rate appears to be less than one inch per year, which we are rounding up to 1.0 inch per year. Although the bluff toe will still be partially protected by the sandy beach, the retreat rate should increase to approximately 2.5 inches per year after 2070 as the bluff toe is continually subject to strong wave activity.

Cumulative sea level rise is from 2020. Table 2 sums up the amount of projected retreat using estimated retreat rates over a 75-year span from a time of 2020 construction. This results in a total bluff retreat of 12 feet.

Table 2: Bluff Retreat Rate				
Years	Span (years)	Cumulative Sea Level Rise (inches)	Retreat Rate (inches per year)	Amount of Retreat (inches)
2020-2030	10	10"	1.0"/yr.	10
2030-2050	20	23"	1.5"/yr.	30
2050-2070	20	42"	2.0"/yr.	40
2070-2095	25	75"	2.5"/yr.	63
				143" = 12'

5.6 Tsunami Hazard

As typical of the Mendocino County coastal area, the site could be subject to large storm waves or tsunami waves. In February 1960, the Point Cabrillo Light House was damaged by an approximately 60 feet high storm wave (meteorological tsunami, or "meteotsunami"). No such waves are recorded at the light house from 1909, the year it was built, to 1960. Nor have such large waves occurred since 1960. BAI is not aware of any such large waves that may have occurred in Little River in February 1960, or at any other time. Since the bluffs are



approximately 40 to 45 feet in vertical height, impact or inundation from a severe storm surge or tsunami event is considered a low (but nonetheless real) risk for the site.

Tsunamis are caused by large-scale sea floor elevation changes resulting from earthquakes on thrust faults associated with tectonic subduction zones. Major earthquakes have occurred along these Pacific Rim subduction zones in recent times; however, no significant tsunami in the Mendocino coastal zone has resulted from these earthquakes. Tsunami damage has been limited to boats and docks within the coves and harbors in Mendocino County. There are several factors that minimize the tsunami potential for Mendocino County:

- The San Andreas Fault is a strike slip fault. Earthquake fault rupture causes ground shifting relative to one side versus the other, but does not result in large, vertical uplift.
- The Mendocino Escarpment is a large, undersea ridge that extends west of Cape Mendocino. The ridge forms a partial wall that runs a few hundred miles to the west. According to Trenkwalder and Stover, the overall effect is that tsunami waves running south toward the escarpment tend to turn north “to impinge on Crescent City”.
- In the area south of the Mendocino Escarpment, the ocean is deeper than in the region north of the escarpment. This effect causes a dispersion and reduction in tsunami wave energy in the coastal waters south of Cape Mendocino.

5.7 Seismicity and Faulting

As is typical of the Mendocino County area, the site will be subject to strong ground shaking during future, nearby, large magnitude earthquakes originating on the active San Andreas Fault, Maacama fault, or possibly other, more distant fault systems. The intensity of ground shaking at the site will depend on the distance to the causative earthquake epicenter, the magnitude of the shock, and the response characteristics of the underlying earth materials. Generally, structures founded in supporting materials and designed in accordance with current building codes are well suited to resist the effects of ground shaking.

No evidence of recent faulting was observed by BAI or shown in the site vicinity on the published geologic maps that we reviewed for this investigation. The presence of ancient faults within the coastal bluffs is common, and should not impact the proposed residence due to their inactivity. Therefore, the potential for fault rupture at the site is considered low.

5.8 Slope Stability Analysis

Our bluff stability analysis was performed to correspond, as a minimum, to the guidelines by California Coastal Commission, “Establishing Development Setbacks from Coastal Bluffs”, Proceedings, California and the World Ocean '02. The document recommends a factor of safety greater than or equal to 1.5 for static conditions and 1.1 for seismic conditions and a horizontal seismic coefficient of 0.15.

We also followed the guidelines prepared by (1) American Society of Civil Engineers (ASCE) and Southern California Earthquake Center (SC/EC) “Recommended Procedures for Implementation of Division of Mines and Geology Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California”, dated June 2002 and (2) California



Geological Survey (CGS) “Guidelines for Evaluating and Mitigating Seismic Hazards in California” dated 2008.

Cross Section A-A’, Plate 15, was created from elevations obtained from Google Earth, our reconnaissance and data from our subsurface exploration as well as subsurface exploration on neighboring properties. The location of the cross section used for our stability analyses is shown on Plate 2.

From our subsurface exploration, six soil and bedrock units, with different density and strength parameters, were identified within the bluff for our stability analyses. Unit “1” is the existing topsoil and silty sand (terrace deposits) that are loose. Unit “2” is the silty sand (lower terrace deposits) that are medium dense. Unit “3” is the deeply weathered bedrock. Unit “4” is the moderate weathered bedrock. Unit “5” is the little weathered bedrock. Unit “6” is the underlying hard bedrock. Table 3 summarizes soil and bedrock parameters used.

Table 3: Soil and Bedrock Parameters			
Unit	Wet Density (pcf)	Cohesion (psf)	Friction Angle (ϕ)
1	105	100	0
2	120	200	30
3	125	600	0
4	125	600	0
5	135	1,100	0
6	140	2,800	0

The above assigned strengths were determined from strength test results obtained from this site and adjacent sites, as well as from back-analysis of the slope stability calculations. The stability of the bluff slope was analyzed using the computer program SLIDE 5.0 version 5.044 by Rocscience, Inc. The results of our stability analyses are presented in Appendix B.

The results of our stability analyses indicate the bluff is stability with a factor of safety that is greater than 1.5 for static conditions and a factor of safety greater than 1.1 for pseudo-static conditions using a seismic coefficient of 0.15.

5.9 Liquefaction

To evaluate liquefaction¹ potential, we performed laboratory testing of the soils and a liquefaction analysis. The results of our analysis indicate the potential for liquefaction at the site during a design earthquake moderate. This analysis was based on procedures by Idriss and Boulanger, 2008, with 2014 update.

Where the factor of safety was 1.3 or less for liquefaction potential, we performed an analysis to estimate induced vertical settlement and lateral spreading due to liquefaction. Liquefaction analysis results are presented in Appendix C.

¹ Liquefaction results in a loss of shear strength and potential soil volume reduction in saturated sandy, silty, silty/clayey, and also coarse gravelly soils below the groundwater table from earthquake shaking. The occurrence of this phenomenon is dependent on many factors, including the intensity and duration of ground shaking, the soil age, density, particle size distribution, and position of the groundwater table.



Lateral spreading is generally caused by liquefaction of marginally stable soils underlying gently to steeply-inclined slopes. In these cases, the saturated soils move toward an unsupported face, such as a roadway cut or bluff face.

The results of our analysis for liquefaction induced settlement and lateral spreading are shown in the following table. For depth of layers that have the potential of liquefaction see boring logs and analysis.

Table 4: Liquefaction Induced Settlement and Lateral Spreading		
Boring	Settlement (inches)	Lateral Spreading (inches)
B-1	0.0	0.4
B-2	0.0	0.0
B-3	1.5	8.6

To mitigate the concern of liquefaction, the planned structure should be supported on drilled piers or spread footings into competent supporting soils or bedrock. Recommendations for the foundations are presented in section 6.0 of this report.

6.0 RECOMMENDATIONS

6.1 Setbacks

Based on our aerial photograph analysis and project erosion rates (Table 2), we have determined a projected retreat of approximately 12 feet over the next 75 years. Using a safety factor of 3, the resulting bluff setback would be 36 feet. Our stability analysis indicates that the bluff is stable for both static and seismic, so no additional setback is required due to slope stability.

6.2 Site Grading

6.2.1 Clearing and Stripping

Areas to be graded should be cleared of existing vegetation, rubbish, and debris. After clearing, surface soils that contain organic matter should be stripped. In general, the depth of required stripping will be about 4 to 6 inches; deeper stripping and grubbing may be required to remove stumps and concentrations of organic matter or roots. The cleared materials should be removed from the site; however, strippings can be stockpiled for later use in landscape areas.

6.2.2 Structural Area Preparation

Within the building area and non-structural-supported slab areas, weak and porous soils should be removed to a depth of 12 to 36 inches, as determined by BAI. Removal of these soils should extend for a minimum distance of about 5 feet beyond the perimeter of the building foundation and about 3 feet beyond exterior concrete slabs. BAI should observe the soils exposed by the recommended excavations. The depth of over excavation should also allow for at least 24 inches



of compacted fill under planned foundations and concrete slab-on-grade. Within building and exterior slab areas, a geotextile stabilization fabric, such as Mirafi HP Series or equivalent, should be used on the bottom of the excavation to help minimize liquefaction and densification differential settlement. Prior to fabric placement, the exposed soils should be scarified to about six inches deep; moisture conditioned to at least optimum moisture content and compacted to at least 90 percent relative compaction as determined by the ASTM D 1557 test procedure, latest edition.

Fill material, either imported or on-site, should be free of perishable matter and rocks greater than six inches in largest dimension, and should be approved by a representative of BAI before fill placement. We anticipate that the existing on-site soils to be excavated, in a "cleaned" condition (i.e., less any organics and debris) are satisfactory for reuse as compacted fill. Imported fill for use in structural areas should be of relatively low expansion potential (i.e., Expansion Index of 40 or less).

Low-expansive engineered fill, on-site or imported, should be placed in thin lifts, six to eight inches depending on compaction equipment, moisture conditioned to near optimum moisture content and compacted to at least 90 percent relative compaction to achieve planned grades.

6.3 Foundation Support

6.3.1 General

As encountered in our test borings, most of the building area is underlain by approximately one to 11 feet of weak soils or soils that have a liquefaction potential. These soils are unsuitable for foundation support in their current state. Structure foundations and concrete slabs placed directly upon weak or porous soils could undergo damaging differential settlement due to porous soil collapse when loaded in a saturated condition. Foundation-supporting elements must penetrate through these weak soils using drilled piers or, a compacted fill pad can be constructed for footing support. The foundations for the residence should be completely in compacted fill or supporting bedrock; foundations should not be supported on different materials (soil to bedrock).

6.3.2 Drilled Piers

Support for the residence can be obtained using cast-in-drilled-hole, reinforced-concrete piers interconnected with grade beams. Drilled piers should be at least 18 inches in diameter and should be embedded a minimum of five feet into bedrock below the liquefaction potential zones. The depth to bedrock varies from one to 11 feet at our boring locations. The average minimum pier depth is anticipated to be approximately 6 to 16 feet below the ground surface. Pier length and diameter should be determined by a structural engineer based on our recommendations.

Pier spacing should be no closer than three pier diameters, center to center. The drilled piers should be designed to gain support from skin friction. A skin friction value of 450 pounds per square foot (psf) of shaft area may be used in the bedrock, for dead loads plus live loads. For total downward loads due to wind or seismic forces, the pier capacity can be increased by one third. Both downward and uplift frictional capacity should be neglected in the potential liquefaction zone.



Resistance to lateral loads can be obtained using passive earth pressure against the face of the foundations. An allowable passive pressure of 350 psf per foot of depth can be used within the bedrock. Passive pressure should be neglected within and above the potential liquefaction zone. Passive pressure can be projected over two pier diameters, however, should not be used below depths of about 8 pier diameters from top of piers.

During bidding, we recommend that proposed drillers be given a copy of this report to review. Caving was not encountered in our borings, however the driller should be prepared to case pier holes if and where caving occurs. Practical drilling refusal with the drill rig was encountered within our boring B-2 at 14.5 feet. With a majority of the building area on the sea stack, difficult drilling conditions should be anticipated. The drilling contractor should be prepared to use rock-coring equipment to achieve full depth, if necessary.

Where final pier depths have been achieved, as determined by BAI, the bottoms of the pier holes should be cleaned of loose materials. Final clean out of the pier holes should be observed by BAI. Concrete should not be placed free fall or in such a manner as to hit the sidewalls of the pier hole.

Groundwater can be shallow or close to the surface during and following periods of rain. If necessary, pier holes should be dewatered prior to placement of reinforcing steel and concrete. If pumping is not practical, concrete should be tremied into place with an adequate head to displace water or slurry, if groundwater has entered the pier hole.

6.3.3 Spread Footings

The residence can be supported on reinforced concrete footings founded in compacted fill, with at least 24 inches of compacted soil below the bottom of footings, placed in accordance with our recommendations. The residence should be supported on either drilled piers or spread footing, but not both. Footings founded in compacted fill can be designed using an allowable soil bearing pressure of 2000 pounds per square foot (psf) for dead plus live loads. A 33 percent increase in bearing pressure is allowable for total loads, including wind or seismic loads. The spread footings should be designed to span a distance of at least five feet of unsupported footing due to the potential for liquefaction differential settlement.

Footing elements should be founded at least 12 inches below lowest adjacent finish grade or 18 inches for two story construction. Wall footings should be no less than 12 and 15 inches wide for one and two story construction, respectively. No subsurface structures (such as subsurface walls, tanks, other foundations, or utility lines) should extend below the footings, or within a zone defined by a 45-degree angle projected downward from the outside, bottom edges of the footings. Completed foundation excavations should be observed by BAI prior to the placement of reinforcing steel.

Resistance to lateral loads can be obtained using passive earth pressure against the face of the foundations. An allowable passive pressure of 350 psf per foot of depth below subgrade and frictional resistance of 0.30 times net vertical dead load, are appropriate for footing elements poured neat against supporting approved engineered fill soils, if required. Passive pressure



should be neglected within the upper six inches of subgrade soils, unless slabs or parameters confine the surface.

6.4 Seismic Design Criteria

The structures should be designed and/or constructed to resist the effects of strong ground shaking (on the order of Modified Mercalli Intensity IX) in accordance with current building codes. The California Building Code (CBC) 2019 edition indicates that the site classification for the property is Site Class F, due to the potential of liquefaction. BAI is anticipating that the fundamental period of vibration will be equal to or less than 0.5 seconds, for which a site-response analysis is not required in accordance with ASCE 7-16. However, if the structural engineer determines that the fundamental period of vibration is greater than 0.5 seconds, BAI will need to re-evaluate the site and may need to perform a site response analysis. Based on a site response analysis not being required, BAI is using Site Class D for design. Accordingly, CBC indicates that the following seismic design parameters are appropriate for the site:

Table 5: Seismic Design Parameters		
Site Class	=	C
Mapped Spectral Response Acceleration at 0.2 sec	$S_s =$	1.945g
Mapped Spectral Response Acceleration at 1.0 sec	$S_1 =$	0.801g
Modified Spectral Response Acceleration at 0.2 sec	$S_{MS} =$	1.945g
Modified Spectral Response Acceleration at 1.0 sec	$S_{M1} =$	1.362g
Design Spectral Response Acceleration at 0.2 sec	$S_{DS} =$	1.297g
Design Spectral Response Acceleration at 1.0 sec	$S_{D1} =$	0.908g
Site Coefficient	$F_a =$	1.0
Site Coefficient	$F_v =$	1.7
Seismic Design Category	=	E

6.5 Concrete Slab-on-Grade

If a structural concrete slab is used (i.e., the slab is supported by and able to span between, interconnecting foundation elements without gaining support from underlying soil), then over-excavation of the near-surface weak soil zone is not required. However, topsoils containing organics should be removed beneath the planned slab (as much as four inches to six inches in depth below existing ground surface).

The weak soils in their present condition are not suitable for slab support. Concrete slab-on-grade floors not supported by foundation elements should be supported on properly compacted fill soils placed in accordance with our recommendations previously presented in Section 6.1 Site Grading.

During foundation and utility trench construction, previously compacted subgrade surfaces may be disturbed. Where this is the case, the subgrade should be moisture conditioned as necessary, and re-rolled to provide a firm smooth unyielding surface compacted to at least 90 percent relative compaction before construction of slabs-on-grade.



Interior slab-on-grade floors should be underlain by at least 4 inches of clean, free-draining crushed rock, graded in size from 3/4 maximum to 1/4 inches minimum, to act as a capillary moisture break. Within traffic or vibratory loaded areas, crushed material should be used to provide a tight interior lock for the aggregates. An underslab drain should be constructed as shown on Plate 16. Where migration of moisture through the floor slab would be detrimental to its intended use, install a vapor retarder membrane. The membrane, placed upon the gravel/crushed rock layer, should be at least 15 mils thick (i.e., Stego® Wrap 15-mil Class A, Carlisle RMB 400 15-mil Class A, or equivalent), and installed in accordance with the manufacturer's specifications to prevent moisture migration through the seams. With a 15-mil minimum thickness membrane, the 2 inches of wetted sand typically placed upon the membrane may be omitted. Construction of moisture/vapor retarders does not guarantee the prevention of moisture moving through the floor slab.

6.6 Retaining Walls

Subsurface or retaining walls should be provided with permanent back drainage to prevent buildup of hydrostatic pressure. Drainage and backfill details are presented on Plate 17. In areas where movement of moisture/vapor through the wall would be detrimental to its intended use, installation of a vapor retarder membrane should be considered. Construction of vapor retarders does not guarantee the prevention of moisture moving through concrete walls. Retaining walls should be designed to resist the lateral earth pressures presented on Plate 18. If a wall drain is not practical, subsurface walls should be designed for full hydrostatic pressure.

In addition to static loads, the retaining walls should also be designed to resist potential seismic loads, in accordance with CBC requirements. For seismic loads, a pressure increment equivalent to an inverted triangular distribution is recommended, varying from 0 (zero) pounds per square foot (psf) at the top of the wall to $23H$ psf at the bottom of the embedded portion, where “H” is the height of the embedded portion (resultant dynamic thrust act at $1/3H$ above the base of the wall).

Quality, placement and compaction requirements for backfill behind subsurface walls are the same as previously presented for fill. Light compaction equipment should be used near the wall to avoid overstressing the walls.

6.7 Site Drainage

Because surface and/or subsurface water is often the cause of foundation or slope stability problems, care should be taken to intercept and divert concentrated surface flows and subsurface seepage away from the building foundations and the bluff edge. Roof runoff water should be directed away from the residence and dispersed, as much as practical, across the lot. Drainage across the lot should be by sheet-flow. Surface grades should maintain a recommended five percent gradient away from building foundations.

If a raised wood floor is used, the area under the floor should be graded to drain towards an under house drain with a conduit outlet(s) through the footings/stem walls. Two-inch or four-inch PVC sleeves, or equivalent should be placed within the forms, at the lowest grade within the



crawl space, prior to concrete placement. Drain pipe should be extended to an area where crawl space can drain through the gravity drain.

Irrigation near the bluff edge should be kept to a minimum. Saturation of these weak soils, or excess seepage along their base, could cause sloughing and accelerated bluff edge retreat. Care should be taken to avoid concentrated surface flow of runoff along the bluff edge.

7.0 ADDITIONAL SERVICES

Prior to construction, BAI should review the final grading and foundation plans, and geotechnical related specifications for conformance with our recommendations. During construction, BAI should be retained to provide periodic observations, together with the appropriate field and laboratory testing during site preparation, subdrain installations, and placement and compaction of fills. Foundation excavations should be reviewed by BAI while the excavation operations are being performed. Our reviews and tests would allow us to check that the work is being performed in accordance with project guidelines, confirm that the soil and rock conditions are as anticipated, and to modify our recommendations, if necessary.

8.0 LIMITATIONS

This geotechnical investigation and engineering geologic reconnaissance of the ocean bluff property were performed in accordance with the usual and current standards of the profession, as they relate to this and similar localities. No other warranty, expressed or implied, is provided as to the conclusions and professional advice presented in this report. Our conclusions are based upon reasonable geological and engineering interpretation of available data.

The samples taken and tested, and the observations made, are considered to be representative of the site; however, soil and geologic conditions may vary significantly between test borings and across the site. As in most projects, conditions revealed during construction excavation may be at variance with preliminary findings. If this occurs, the changed conditions must be evaluated by BAI, and revised recommendations be provided as required.

This report is issued with the understanding that it is the responsibility of the Owner, or his/her representative, to insure that the information and recommendations contained herein are brought to the attention of all other design professionals for the project, and incorporated into the plans, and that the Contractor and Subcontractors implement such recommendations in the field. The safety of others is the responsibility of the Contractor. The Contractor should notify the owner and BAI if he/she considers any of the recommended actions presented herein to be unsafe or otherwise impractical.

Changes in the condition of a site can occur with the passage of time, whether they are due to natural events or to human activities on this, or adjacent sites. In addition, changes in applicable or appropriate codes and standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, this report may become invalidated wholly or partially by changes outside of our control. Therefore, this report is subject to review and revision as changed conditions are identified.



The recommendations contained in this report are based on certain specific project information regarding type of construction and current building location, which have been made available to us. If conceptual changes are undertaken during final project design, we should be allowed to review them in light of this report to determine if our recommendations are still applicable.



ILLUSTRATIONS





REFERENCE:
Mendocino Quadrangle
7.5-Minute Series, USGS, 2018

Latitude: 39.269665
Longitude: -123.789284



APPROXIMATE SCALE (FEET)



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VICINITY MAP
DYM RESIDENCE
7720 North Highway 1
Little River, California

PLATE
1



Reference:
California Coastal Records project, www.californiacoastline.org, by permission.



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**1979 and 2019 COASTLINE OBLIQUE AERIAL
PHOTOGRAPHS**
DYM RESIDENCE
7720 North Highway 1
Little River, California

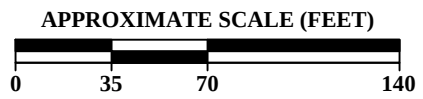
**PLATE
2**

LEGEND

- Boring location and number
- Approximate property line
- Approximate potential building site
- Cross section line
- Photograph location and direction



5/4/2020 11:14:42 AM save date
5/4/2020 11:15:10 AM plot date



REFERENCE:
Potential Building Site, prepared by
Wynn Coastal Planning, dated 10/29/18

	Brunsing Associates, Inc. 5468 Skylane Blvd., Suite 201 Santa Rosa, California 95403 Tel: (707) 528-6108		Job No.: 12934.01	SITE MAP DYM RESIDENCE 7720 North Highway 1 Little River, California	PLATE 3
			Appr.:		
			Date: 05/04/20		

Log of Boring B-1

Equipment: Deeprock DR7KTrack w/ 4-inch solid stem flight auger

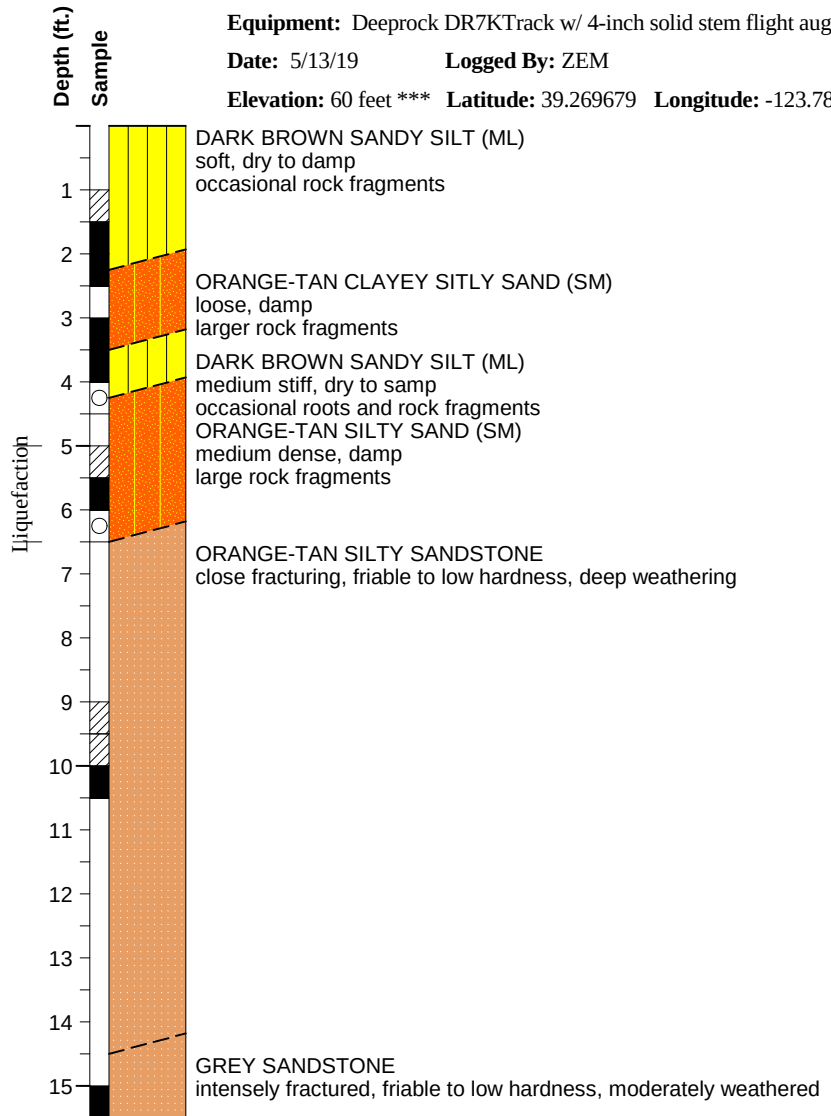
Date: 5/13/19

Logged By: ZEM

Elevation: 60 feet *** Latitude: 39.269679 Longitude: -123.78948

Laboratory Tests

	Sampler Type*	Moisture Content (%)	Dry Density (pcf)	Blows/foot
	CA			7 **
Tx 1174 (576)	CA	30.0	88	7 **
		24.4	97	
	CA			18 **
	CA			24 **
	CM			40/6" **



Notes:

1. No caving
2. No free water encountered
3. Backfilled with cuttings

BORING LOG 1 PER PAGE, 12934.01 GINT.GPJ, 5/4/20

Latitude/Longitude estimated from Google Earth.

* See Soil Classification Chart & Key to Test Data

** Equivalent "Standard Penetration" Blow Counts.

*** Elevations interpolated from Google Earth.



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Date: 05/04/20

LOG OF BORING B-1

DYM RESIDENCE
7720 North Highway 1
Little River, California

Scale: 1" = 3'

PLATE

4

SHEET 1 of 1

Log of Boring B-2

Equipment: Deeprock DR7KTrack w/ 4-inch solid stem flight auger

Date: 5/13/19

Logged By: ZEM

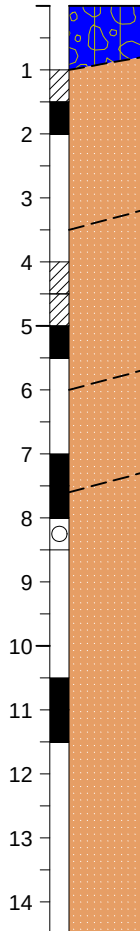
Elevation: 66 feet *** **Latitude:** 39.269558 **Longitude:** -123.789344

Laboratory Tests

Sampler Type*
Moisture Content (%)
Dry Density (pcf)
Blows/foot

Depth (ft.)
Sample

	CA			32/6" **
Tx 1051 (576)	CA	22.2	95	14 **
		27.1	94	
Tx 1210 (1440)	CA	25.7	100	26 **
	CA			38 **



DARK BROWN BROWN SANDY SILTY GRAVEL (GM)
medium dense, dry

RED-ORANGE SANDSTONE
crushed, low hardness to moderate hardness, deeply weathered

RED-ORANGE SANDSTONE
crushed, low hardness to moderate hardness, deeply weathered

ORANGE-YELLOW CLAYEY SANDSTONE
intensely fractured, soft to friable, deeply weathered

ORANGE SILTY SANDSTONE
intensely fractured, soft to low hardness, deeply weathered

Notes:

1. No caving
2. No free water encountered
3. Practical drilling refusal encountered at 14.5 feet
4. Backfilled with cuttings

BORING LOG 1 PER PAGE, 12934.01 GINT.GPJ, 5/4/20

Latitude/Longitude estimated from Google Earth.
* See Soil Classification Chart & Key to Test Data
** Equivalent "Standard Penetration" Blow Counts.
*** Elevations interpolated from Google Earth.

Scale: 1" = 3'



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LOG OF BORING B-2
DYM RESIDENCE
7720 North Highway 1
Little River, California

PLATE
5

SHEET 1 of 1

Log of Boring B-3

Equipment: Deeprock DR7KTrack w/ 4-inch solid stem flight auger

Date: 5/13/19

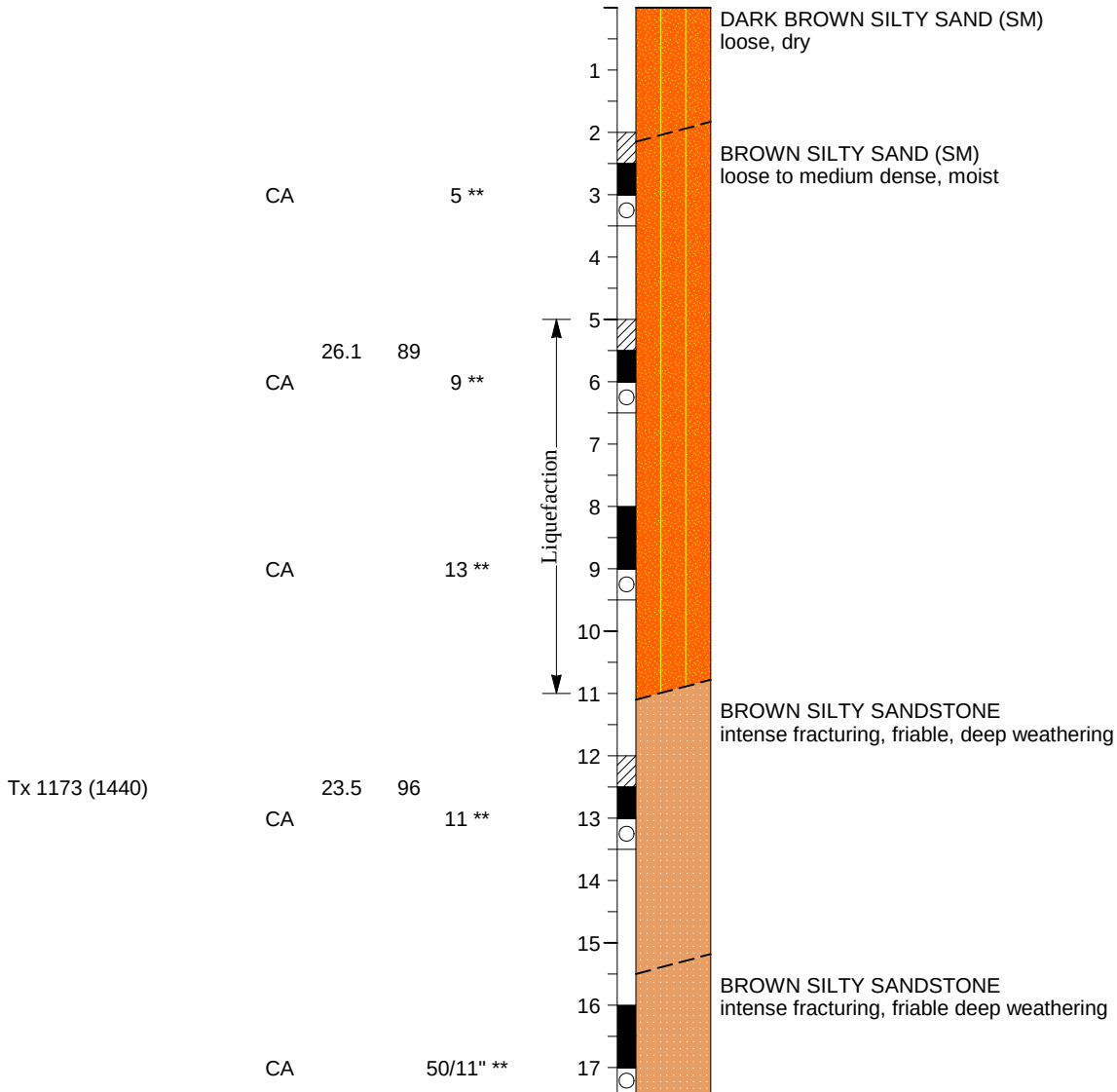
Logged By: ZEM

Elevation: 79 feet *** Latitude: 39.269566 Longitude: -123.789106

Laboratory Tests

Sampler Type*
Moisture Content (%)
Dry Density (pcf)
Blows/foot

Depth (ft.)
Sample



BORING LOG 1 PER PAGE, 12934.01 GINT.GPJ, 5/4/20

Latitude/Longitude estimated from Google Earth.
* See Soil Classification Chart & Key to Test Data
** Equivalent "Standard Penetration" Blow Counts.
*** Elevations interpolated from Google Earth.

Scale: 1" = 3'



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LOG OF BORING B-3
DYM RESIDENCE
7720 North Highway 1
Little River, California

PLATE
6

SHEET 1 of 1

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)	MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS	
				GRAPHIC	LETTER		
	COARSE-GRAINED SOILS	GRAVELS AND GRAVELLY SOILS	CLEAN GRAVELS (Less than 5% fines)		GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	
					GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	
			GRAVELS WITH FINES (Greater than 12% fines)		GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES	
					GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES	
		SAND AND SANDY SOILS	CLEAN SANDS (Less than 5% fines)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	
					SP	POORLY-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	
			50% OR MORE OF COARSE FRACTION PASSING THROUGH NO. 4 SIEVE	SANDS WITH FINES (Greater than 12% fines)		SM	SILTY SANDS, SAND-SILT MIXTURES
						SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE-GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT 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 GREATER THAN 50		MH
		CH	INORGANIC CLAYS OF HIGH PLASTICITY				
		OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS				
		PT	PEAT, HUMOUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS				
	HIGHLY ORGANIC SOILS						

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

KEY TO TEST DATA

LL - Liquid Limit	Consol - Consolidation	Shear Strength, psf	Confining Pressure, psf
PI - Plasticity Index	EI - Expansion Index	Tx 1564 (1440) - Unconsolidated Undrained Triaxial	
■ Sample Retained	SA - Sieve Analysis	TxCU 1564 (1440) - Consolidated Undrained Triaxial	
▨ Sample Recovered, Not Retained		DS 2020 (1440) - Consolidated Drained Direct Shear	
⊠ Bulk Sample		FVS 520 - Field Vane Shear	
□ Sample Not Recovered		UC 1500 - Unconfined Compression	
CA - California Modified Split Barrel Sampler 3.0-inch O.D.		PP 1500 - Field Pocket Penetrometer	
CM - California Modified Split Barrel Sampler 2.5-inch O.D.		Sat - Sample saturated prior to test	
SPT - California Split Barrel Sampler 2.0-inch O.D.			
SH - Shelby Tube			▽ Initial Groundwater Level Reading
RC - Rock Coring			▼ Second Groundwater Level Reading
Recovery - Percent Core Recovered			
RQD - Rock Quality Designation (length of core pieces >= 4-inches / core length)			



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SOIL CLASSIFICATION CHART & KEY TO TEST DATA

DYM RESIDENCE
7720 North Highway 1
Little River, California

PLATE
7

RELATIVE DENSITY OF COARSE-GRAINED SOILS

Relative Density	Standard Penetration Test Blow Count (blows per foot)
Very loose	4 or less
Loose	5 to 10
Medium dense	11 to 30
Dense	31 to 50
Very dense	More than 50

CONSISTENCY OF FINE-GRAINED SOILS

Consistency	Identification Procedure	Approximate Shear Strength (psf)
Very soft	Easily penetrated several inches with fist	Less than 250
Soft	Easily penetrated several inches with thumb	250 to 500
Medium stiff	Penetrated several inches by thumb with moderate effort	500 to 1000
Stiff	Readily indented by thumb, but penetrated only with great effort	1000 to 2000
Very stiff	Readily indented by thumb nail	2000 to 4000
Hard	indented with difficulty by thumb nail	More than 4000

NATURAL MOISTURE CONTENT

Dry	No noticeable moisture content. Requires considerable moisture to obtain optimum moisture content* for compaction.
Damp	Contains some moisture, but is on the dry side of optimum.
Moist	Near optimum moisture content for compaction.
Wet	Requires drying to obtain optimum moisture content for compaction.
Saturated	Near or below the water table, from capillarity, or from perched or ponded water. All void spaces filled with water.

* Optimum moisture content as determined in accordance with ASTM Test Method D1557, latest edition.

Where laboratory test data are not available, the above field classifications provide a general indication of material properties; the classifications may require modification based upon laboratory tests.



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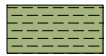
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SOIL DESCRIPTIVE PROPERTIES

DYM RESIDENCE
7720 North Highway 1
Little River, California

PLATE
8

Generalized Graphic Bedrock Symbols



Claystone



Siltstone



Tuff (Volcanic Ash)



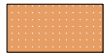
Shale



Chert



Andesite



Sandstone



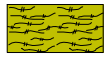
Serpentine



Basalt



Conglomerate



Greenstone



Schist

Stratification

Bedding of Sedimentary Rocks

Massive
Very thick bedded
Thick bedded
Thin bedded
Very thin bedded
Laminated
Thinly laminated

Thickness of Beds

No apparent bedding
Greater than 4 feet
2 feet to 4 feet
2 inches to 2 feet
0.5 inches to 2 inches
0.125 inches to 0.5 inches
less than 0.125 inches

Fracturing

Fracturing Intensity

Little
Occasional
Moderate
Close
Intense
Crushed

Fracture Spacing

Greater than 4 feet
1 foot to 4 feet
6 inches to 1 foot
1 inch to 6 inches
0.5 inches to 1 inch
less than 0.5 inches

Strength

Soft
Friable
Low hardness
Moderate hardness
Hard
Very hard

Plastic or very low strength.
Crumbles by hand.
Crumbles under light hammer blows.
Crumbles under a few heavy hammer blows.
Breaks into large pieces under heavy, ringing hammer blows.
Resists heavy, ringing hammer blows and will yield with difficulty only dust and small flying fragments.

Weathering

Deep	Moderate to complete mineral decomposition, extensive disintegration, deep and thorough discoloration, many extensively coated fractures.
Moderate	Slight decomposition of minerals, little disintegration, moderate discoloration, moderately coated fractures.
Little	No megascopic decomposition of minerals, slight to no effect on cementation, slight and intermittent, or localized discoloration, few stains on fracture surfaces.
Fresh	Unaffected by weathering agents, no disintegration or discoloration, fractures usually less numerous than joints.



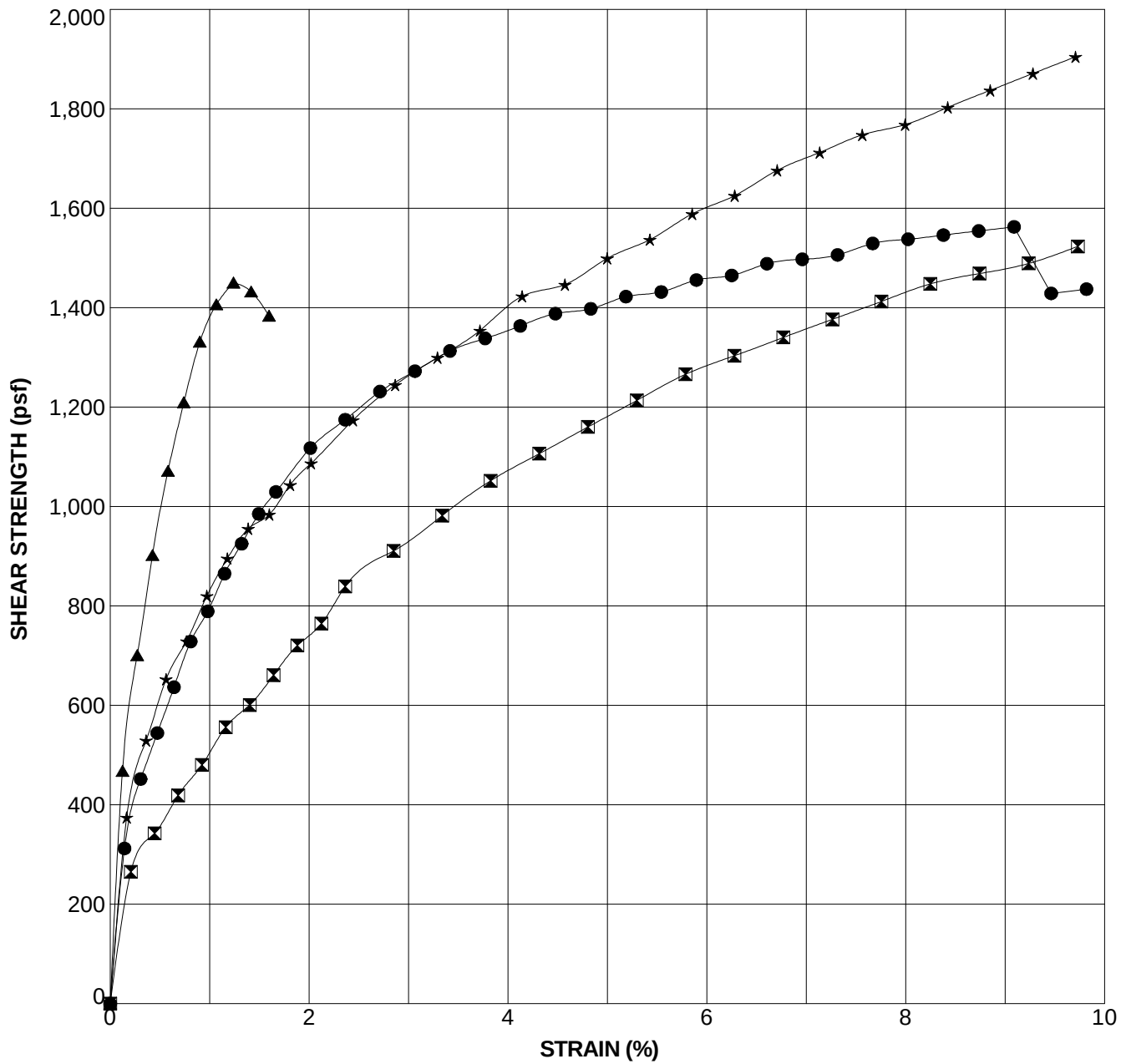
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BEDROCK DESCRIPTIVE PROPERTIES

DYM RESIDENCE
7720 North Highway 1
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PLATE
9



Sample Source	Classification	Confining Pressure (psf)	Yield Strength (psf)	Strain (%)	Dry Density (pcf)	Moisture Content (%)
● B-1 at 4 ft	ORANGE-TAN SILTY SAND (SM)	576	1174	2.4	88	30.0
☒ B-2 at 5 ft	RED-ORANGE SANDSTONE	576	1051	3.8	95	22.2
▲ B-2 at 8 ft	ORANGE SILTY SANDSTONE	1440	1210	0.7	100	25.7
★ B-3 at 12.5 ft	BROWN SILTY SANDSTONE	1440	1173	2.4	96	23.5

UNCONSOLIDATED TRIAXIAL (YIELD), 12934-01 GINT.GPJ, 5/4/20



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**UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION
TEST RESULTS**

DYM RESIDENCE
7720 North Highway 1
Little River, California

**PLATE
10**



Looking east at proposed building site

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5/4/2020 11:37:09 AM plot date



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SITE PHOTOGRAPH A
DYM RESIDENCE
7720 North Highway 1
Little River, California

PLATE
11



Looking northeast

Site Photograph B



Looking southwest from bluff near building site

Site Photograph C

L:\Geotech Projects\12934.01 Dym, Little R

5/4/2020 11:37:41 AM save date
5/4/2020 11:38:03 AM plot date

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SITE PHOTOGRAPH D

DYM RESIDENCE
7720 North Highway 1
Little River, California

PLATE

13



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5/4/2020 11:41:35 AM plot date



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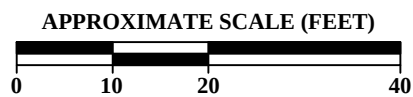
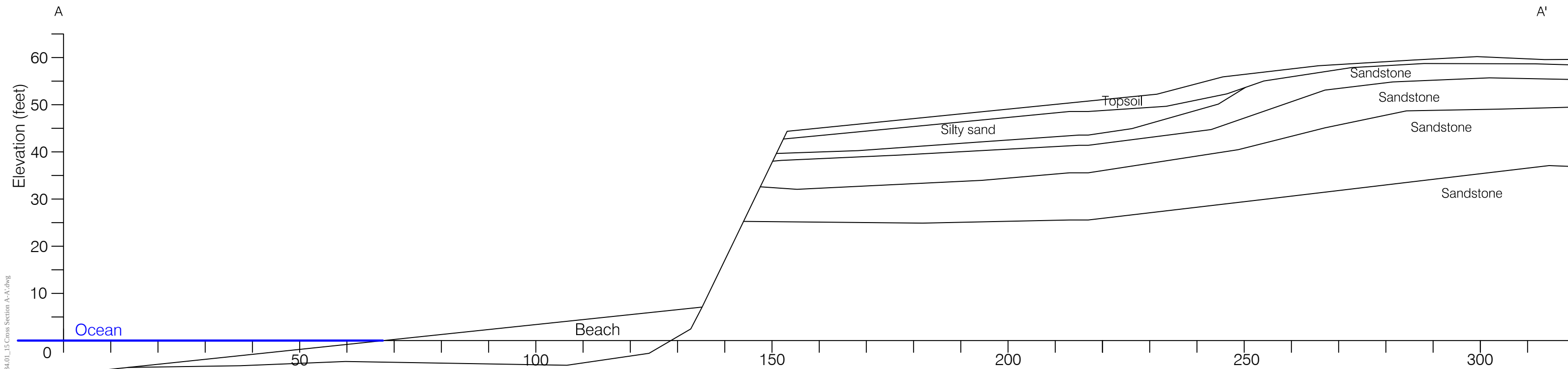
Job No.: 12934.01
Appr.: **EEO**
Date: 05/04/20

SITE PHOTOGRAPH E
DYM RESIDENCE
7720 North Highway 1
Little River, California

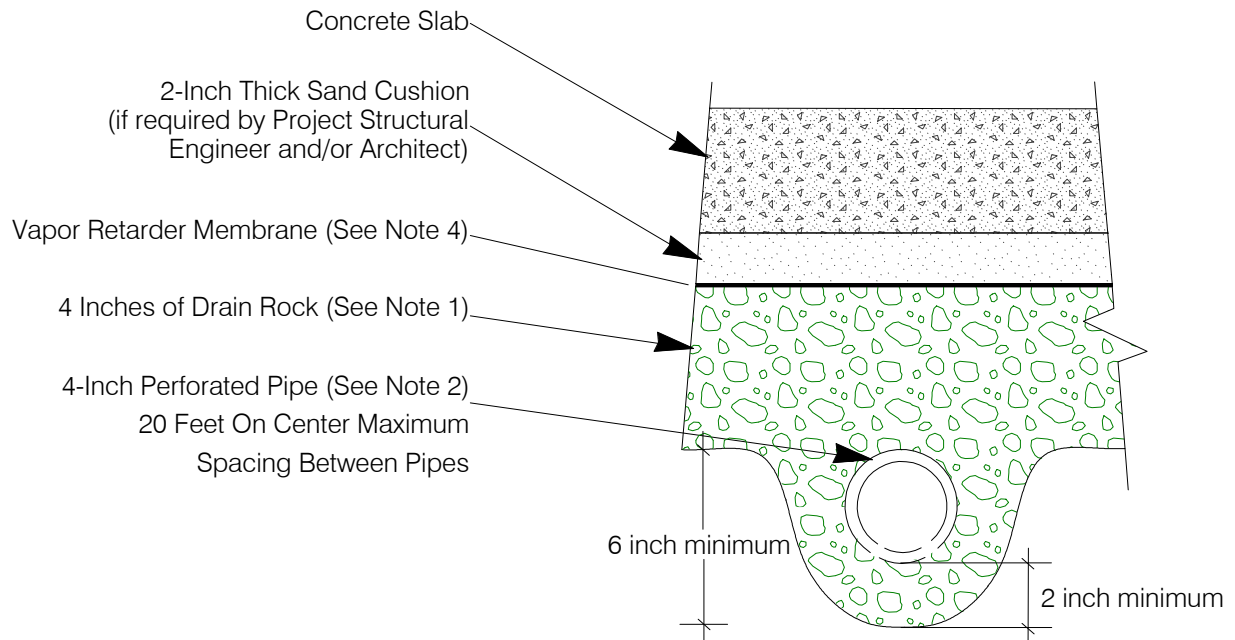
PLATE
14

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5/4/2020 11:42:27 AM plot date



	Brunsing Associates, Inc. 5468 Skylane Blvd., Suite 201 Santa Rosa, California 95403 Tel: (707) 528-6108	Job No.: 12934.01	<u>GEOLOGIC CROSS SECTION A-A'</u> DYM RESIDENCE 7720 North Highway 1 Little River, California	PLATE 15
		Appr.: 		
		Date: 05/04/20		



NOT TO SCALE

NOTES:

1. Drain rock should be clean, free-draining material graded in size between the No.4 and 3/4 inch sieves.
2. Pipe should be SDR 35 or equivalent, perforations placed down, sloped at least 1 percent to gravity outlet, or sump with automatic pump.
3. A clean-out pipe with cap should be installed at the up-slope end of perforated pipe.
4. Vapor retarder should be at least 15-mils thick and installed in accordance with the manufacturer's specifications.

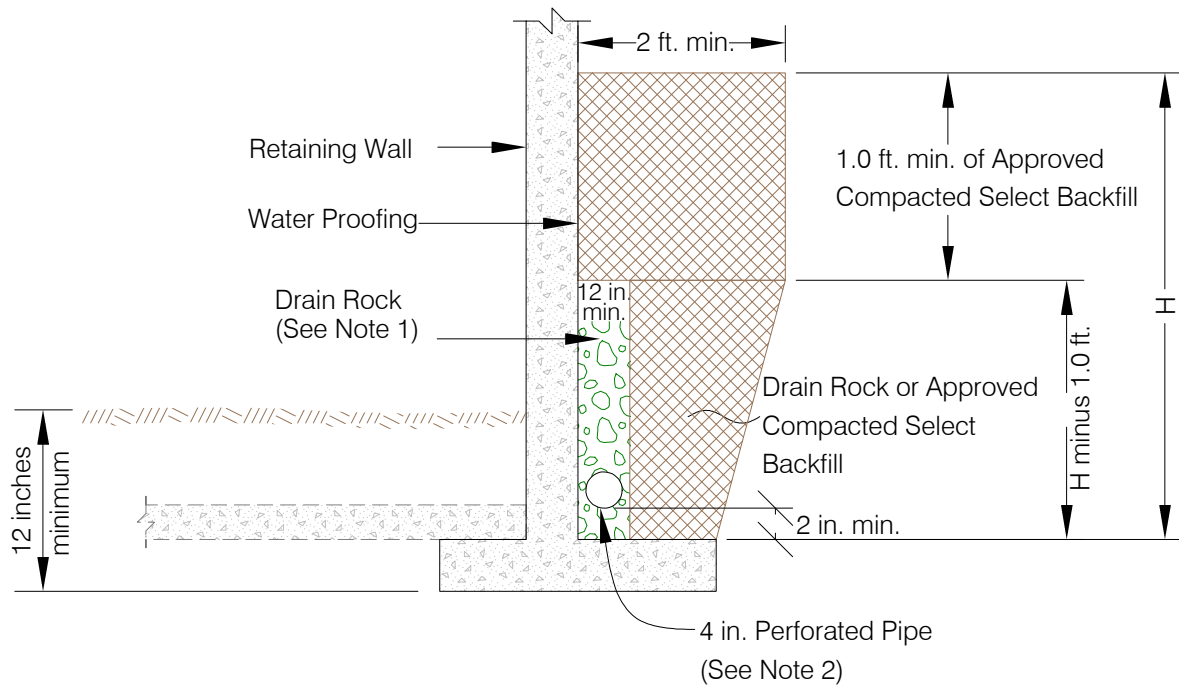


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UNDERSLAB DRAINAGE DETAILS
DYM RESIDENCE
7720 North Highway 1
Little River, California

**PLATE
16**



RETAINING WALL DRAINAGE DETAIL

(Not to Scale)

NOTES:

- (1) Drain rock should be clean, free-draining material graded in size between the No. 4 and 3/4 inch sieves and should be wrapped in a non-woven geotextile filter fabric (Mirafi 140N or equivalent), or Class 2 permeable material, without filter fabric, per Caltrans standard specifications, latest edition.
- (2) Pipe should be SDR 35 or equivalent, placed with perforations down, and sloped at 1 percent to drain to gravity outlet or sump with automatic pump.
- (3) A clean-out pipe with cap should be installed at the up-slope end of perforated pipe, and pipe elbows should be 45 degrees or less (for "snake" access).

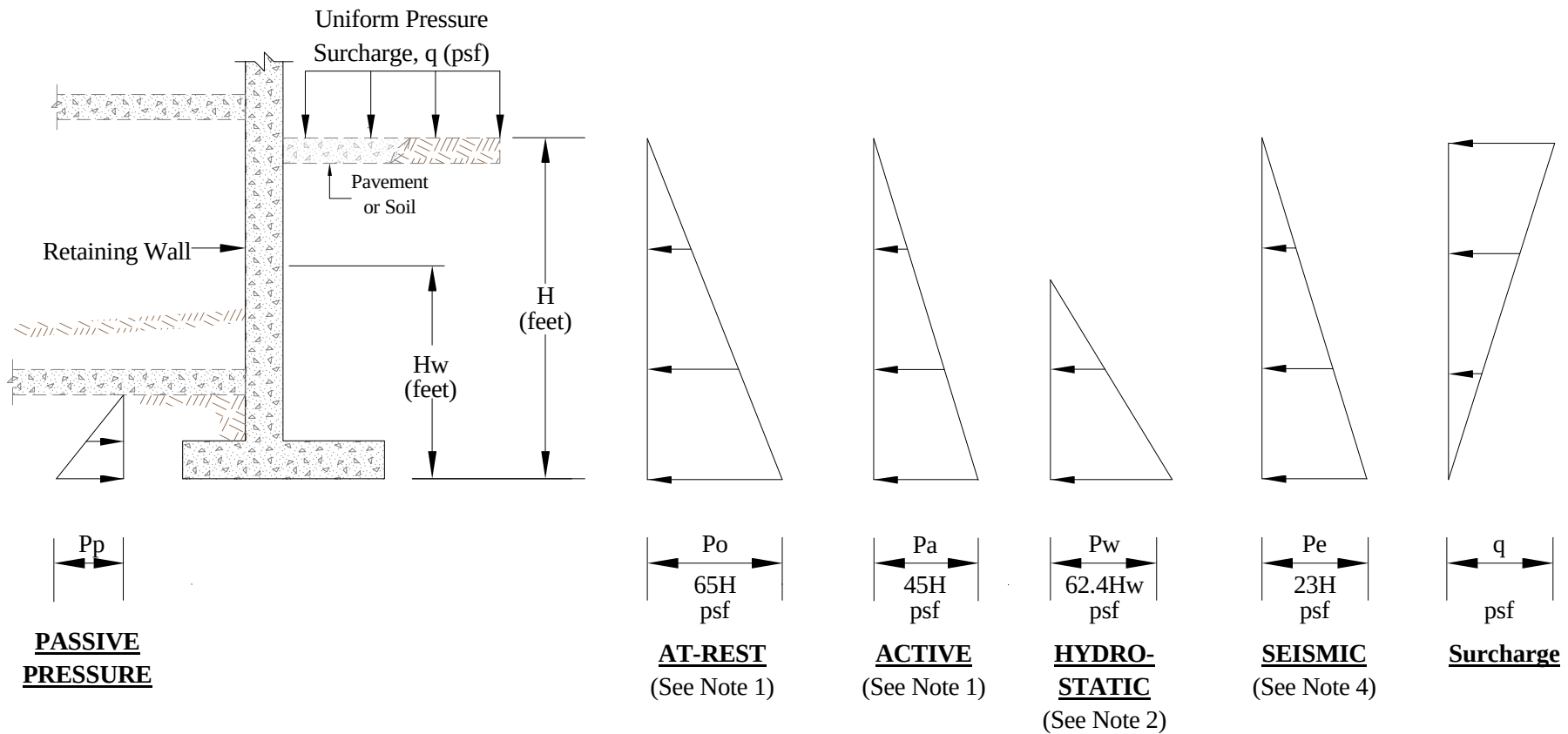


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RETAINING WALL DRAINAGE DETAIL
DYM RESIDENCE
7720 North Highway 1
Little River, California

**PLATE
17**

**NOTES:**

- (1) If the wall at the surface of the backfill cannot move more than about 0.1 percent of its' height, at-rest soil pressures should be used.
- (2) If the wall is drained the above hydrostatic pressure does not have to be used. See Plate 17 for drainage and backfill details.
- (3) The above pressures should be used where backfill slope is flatter than 3 horizontal to 1 vertical (3H:1V). Where backfill slope is between 3H:1V and 1.5H:1V, use active pressure of 55H psf and at-rest pressure of 87H psf, respectively.
- (4) For additional design seismic pressures see the Retaining Walls section of this report.



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RETAINING WALL LATERAL EARTH PRESSURES

DYM RESIDENCE
 7720 North Highway 1
 Little River, California

PLATE
18

APPENDIX A

References

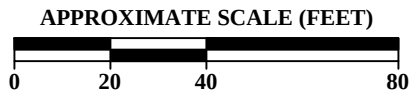
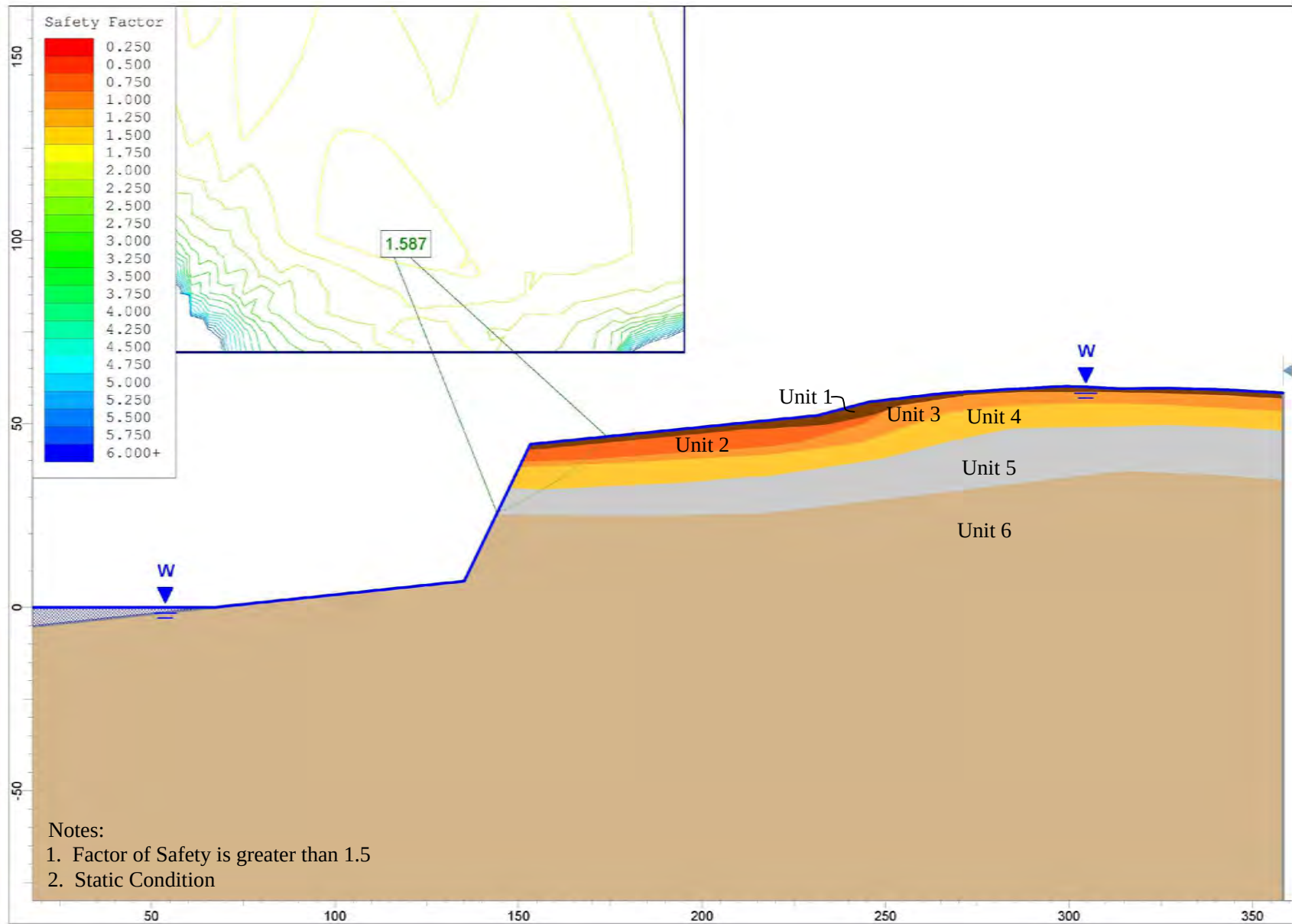
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APPENDIX B

Slope Stability Analysis





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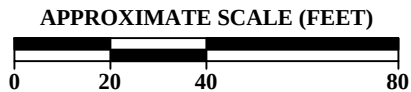
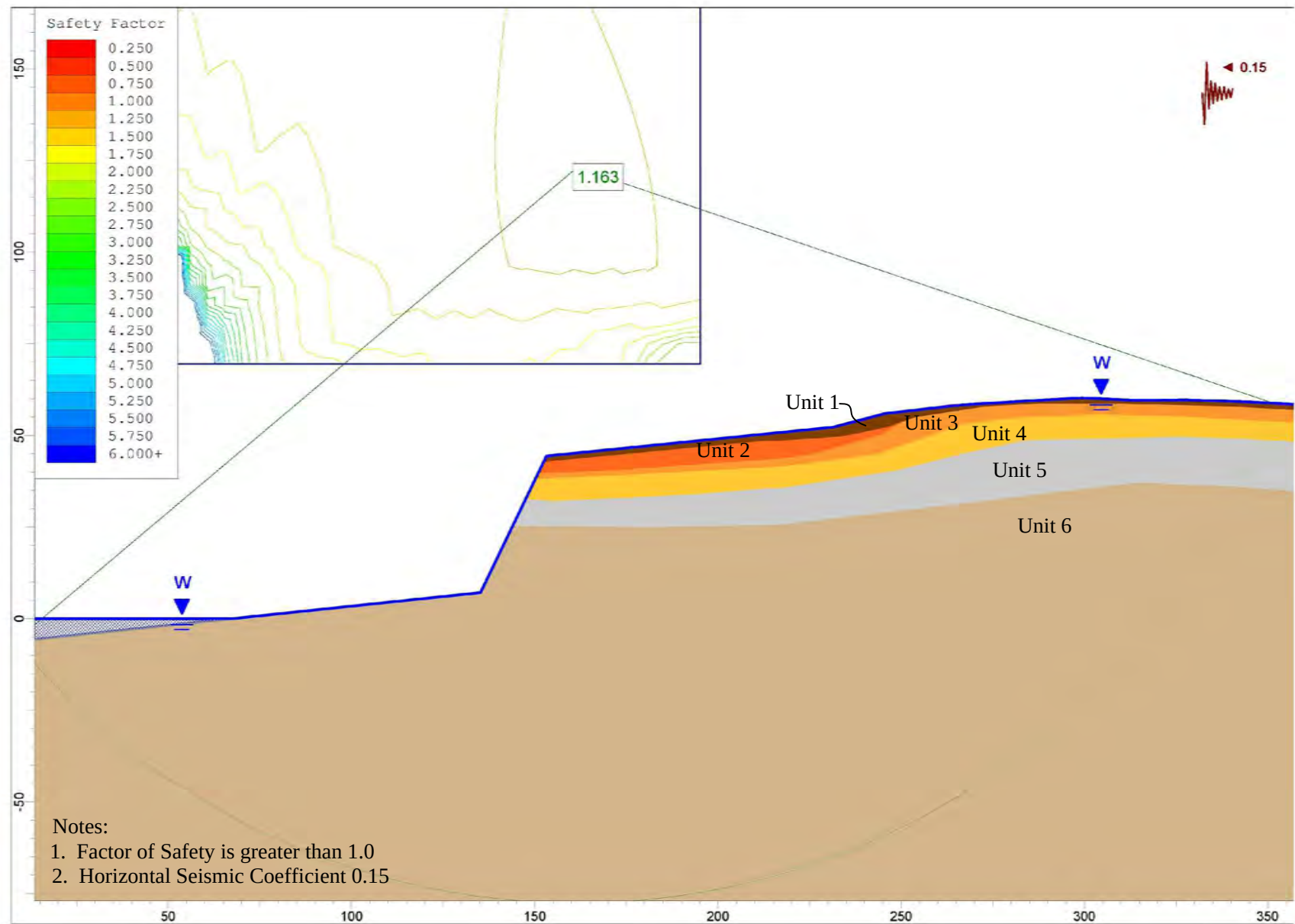
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STATIC SLOPE STABILITY CROSS SECTION A-A'

DYM RESIDENCE
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**PLATE
 B1**



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Date: 05/04/20

SEISMIC SLOPE STABILITY CROSS SECTION A-A'

DYM RESIDENCE
7720 North Highway 1
Little River, California

**PLATE
B2**

APPENDIX C

Liquefaction Analysis



Project: Dym Residence
 Project #: 12934.01
 Date: 5/4/2020
 Boring: B-1

Input Parameters:

Peak Ground Accel (g) = 0.919
 Earthquake Magnitude, M = 7.3
 Water Table Depth (m) = 1.5
 Average γ Above Water Table (kN/m^3) = 18.1
 Average γ Below Water Table (kN/m^3) = 18.9
 Borehole Diameter (mm) = 101.60
 Requires Correction for Sample Liners (YES/NO): no
 Rod Lengths Assumed Equal to the Depth Plus 1.5 m (for the above ground extension)
 Gravity Acceleration (m/sec^2) = 9.81
 Height of Exposed Face (m) = 12
 Boring Distance from Exposed Face (m) = 25

Liquefaction Potential

SPT Sample Number	Depth (m)	Depth (ft)	Measured		Soil Type (USCS)	Flag "inp"	Sat/Unsat	Fines Content (%)		Energy Ratio, ER (%)	C _g , C _r , C _f , C _s				N ₆₀	σ _{vc} (kPa)		C _h	ΔN for Fines Content (N ₁) ₆₀	Stress Reduct. Coeff., F _g	MSF for Sand		CRR for M=7.5 & σ _{vc} =1 atm	Factor of Safety				
			C _E	C _B				C _R	C _F		σ _{vc}	σ _{vc} '	CSR	MSF _{max}		MSF for Sand	CRR											
1	0.67	2.20	7	ML			0	50	75	1.25	1	0.75	1.00	6.6	12	12	1.70	11.16	5.6	16.77	1.00	0.598	1.4	1.03	1.10	0.172	N.A.	N.A.
2	1.07	3.50	7	SM/ML			0	50	75	1.25	1	0.75	1.00	6.6	19	19	1.70	11.16	5.6	16.77	1.00	0.596	1.4	1.03	1.10	0.172	N.A.	N.A.
3	1.52	5.00	7	SM			0	40	75	1.25	1	0.8	1.00	7.0	28	28	1.70	11.90	5.6	17.48	0.99	0.594	1.4	1.03	1.10	0.178	N.A.	N.A.
4	1.98	6.50	18	SM			1	40	75	1.25	1	0.8	1.00	18.0	36	32	1.49	26.90	5.6	32.47	0.99	0.675	2.2	1.08	1.10	0.695	0.82	1.22
5	4.72	15.50	24	Sandstone			1	35	75	1.25	1	0.95	1.00	28.5	88	56	1.19	33.96	5.5	39.47	0.96	0.891	2.2	1.08	1.10	2.000	2	2.00

Liquefaction Potential

Liquefaction Induced Settlement and Lateral Spreading

Depth (m)	Depth (ft)	Limiting Shear Strain γ_{lim}	Parameter E_a	Maximum Shear Strain γ_{max}	ΔH_i (m)	ΔLDI_i (m)	ΔLDI_i (in)	Vertical Reconsol.	
								Strain ϵ_v	ΔS_i (m)
0.67	2.20	0.227	0.677	0.000	0.67	0.000	0.0	0.000	0.00
1.07	3.50	0.227	0.677	0.000	0.40	0.000	0.0	0.000	0.00
1.52	5.00	0.210	0.645	0.000	0.46	0.000	0.0	0.000	0.00
1.98	6.50	0.033	-0.258	0.023	0.46	0.011	0.4	0.004	0.08
4.72	15.50	0.010	-0.764	0.000	2.74	0.000	0.0	0.000	0.00

(1) Flag "nlp" - based on laboratory testing of fine content, plasticity index, liquid limits and potential intact moisture content.

"nlp" - no liquefaction potential

LDI = 0.01 0.4 S= 0.002 0.08
 (m) (in) (m) (in)

Project: Dym Residence
 Project #: 12934.01
 Date: 5/4/2020
 Boring: B-2

Input Parameters:
 Peak Ground Accel (g) = 0.919
 Earthquake Magnitude, M = 7.3
 Water Table Depth (m) = 1.5
 Average γ Above Water Table (kN/m³) = 18.1
 Average γ Below Water Table (kN/m³) = 18.9
 Borehole Diameter (mm) = 101.60
 Requires Correction for Sample Liners (YES/NO): no
 Rod Lengths Assumed Equal to the Depth Plus
 Gravity Acceleration (m/sec²) = 9.81
 Height of Exposed Face (m) = 12
 Boring Distance from Exposed Face (m) = 18

Liquefaction Potential

SPT Sample Number	Depth (m)	Depth (ft)	N	Soil Type (USCS)	Plug "nlp"	Sat/Uns	Fines Content (%)	Energy Ratio, ER (%)	C _E	C _B	C _R	C _S	N ₆₀	C _u (kPa)	C _v (kPa)	C _H	(N ₁) ₆₀	ΔN for Fines Content (N) ₆₀₋₂₈	Stress Reduct. Coeff., E _d	CSR	MSF _{fact}	MSF for Sand	K _{cs} for Sand	CFR for M=7.5 & C _u =14mm	Factor of Safety
1	0.30	1.00	50	GM		0	35	75	1.25	1	0.75	1.00	46.9	6	6	1.70	79.69	5.5	1.00	0.600	2.2	1.08	1.10	2.000	N.A.
2	1.52	5.00	14	Sandstone		0	40	75	1.25	1	0.8	1.00	14.0	28	28	1.63	22.79	5.6	0.99	0.594	1.9	1.06	1.10	0.399	N.A.
3	1.83	6.00	14	Sandstone	nlp	1	40	75	1.25	1	0.8	1.00	14.0	33	30	1.37	N.A.	N.A.	0.99	0.680	--	--	1.10	N.A.	N.A.
4	2.59	8.50	26	Sandstone		1	40	75	1.25	1	0.85	1.00	27.6	48	37	1.33	36.71	5.6	0.98	0.753	2.2	1.08	1.10	2.000	2
5	4.42	14.50	38	Sandstone		1	40	75	1.25	1	0.85	1.00	40.4	82	54	1.18	47.67	5.6	0.96	0.879	2.2	1.08	1.10	2.000	2

Liquefaction Potential

Liquefaction Induced Settlement and Lateral Spreading

Depth (m)	Depth (ft)	Limiting Shear Strain γ_{lim}	Parameter F_{sa}	Maximum Shear Strain γ_{max}	ΔH_i (m)	ΔLDI_i (m)	ΔLDI_i (m)	Vertical Reconsol. Strain ϵ_v	ΔS_i (m)	ΔS_i (m)
0.30	1.00	0.000	-4.674	0.000	0.30	0.000	0.0	0.000	0.000	0.00
1.52	5.00	0.058	0.019	0.000	1.22	0.000	0.0	0.000	0.000	0.00
1.83	6.00	0.000	0.000	0.000	0.30	0.000	0.0	0.000	0.000	0.00
2.59	8.50	0.005	-0.978	0.000	0.76	0.000	0.0	0.000	0.000	0.00
4.42	14.50	0.000	-1.855	0.000	1.83	0.000	0.0	0.000	0.000	0.00
LDI = 0.00 (m) S= 0.000 (m)										0.00 (m)

(1) Flag "nlp" - based on laboratory testing of fine content, plasticity index, liquid limits and potential intact moisture content.
 "nlp" - no liquefaction potential



Project: Dyn Residence
 Project #: 12934.01
 Date: 5/4/2020
 Boring: B-3

Input Parameters:

Peak Ground Accel (g) = 0.919
 Earthquake Magnitude, M = 7.3
 Water Table Depth (m) = 1.5
 Average γ Above Water Table (kN/m^3) = 18.1
 Average γ Below Water Table (kN/m^3) = 18.9
 Borehole Diameter (mm) = 101.60
 Requires Correction for Sample Liners (YES/NO): no
 Rod Lengths Assumed Equal to the Depth Plus 1.5 m (for the above ground extension)
 Gravity Acceleration (m/sec^2) = 9.81
 Height of Exposed Face (m) = 12
 Boring Distance from Exposed Face (m) = 17

Liquefaction Potential

SPT Sample Number	Depth (m)	Measured Depth (ft)	Soil Type (USCS)	Flag "nlp"	Sat/Unsat	Fines Content (%)	Energy Ratio, ER (%)	C_E	C_B	C_R	C_S	N_{60}	C_{sc} (kPa)	C_{sc} (kPa)	C_N	$(N_1)_{60}$	ΔN for Fines Content (N_1) ₆₀	Stress Reduct. Coeff, r_d	CSR	MSF _{max}	MSF for Sand	K_{cs} for Sand	CRR for $M=7.5$ & $C_{vc}=1$ atm	Factor of Safety
1	0.61	2.00	5	SM	0	35	75	1.25	1	0.75	1.00	4.7	11	11	1.70	7.97	5.5	1.00	0.598	1.3	1.02	1.10	0.144	N.A.
2	1.52	5.00	5	SM	0	35	75	1.25	1	0.8	1.00	5.0	28	28	1.70	8.50	5.5	0.99	0.594	1.3	1.02	1.10	0.148	N.A.
3	2.29	7.50	9	SM	1	35	75	1.25	1	0.8	1.00	9.0	43	34	1.61	14.46	5.5	0.99	0.717	1.5	1.03	1.10	0.205	0.33
4	3.35	11.00	13	SM	1	35	75	1.25	1	0.85	1.00	13.8	62	44	1.40	19.27	5.5	0.97	0.819	1.7	1.05	1.10	0.285	0.33
5	5.33	17.50	55	Sandstone	1	30	75	1.25	1	0.95	1.00	65.3	99	62	1.14	74.27	5.4	0.95	0.910	2.2	1.08	1.10	2.000	2

Liquefaction Potential

Liquefaction Induced Settlement and Lateral Spreading

Depth (m)	Limiting Depth (ft)	Limiting Shear Strain γ_{lim}	Parameter F_{eq}	Maximum Shear Strain γ_{max}	ΔH_i (m)	ΔLDI_i (m)	ΔLDI_i (in)	Vertical Reconsol. Strain ϵ_v	ΔS_i (m)	ΔS_i (in)
0.61	2.00	0.324	0.813	0.000	0.61	0.000	0.0	0.000	0.000	0.00
1.52	5.00	0.306	0.793	0.000	0.91	0.000	0.0	0.000	0.000	0.00
2.29	7.50	0.160	0.519	0.160	0.76	0.122	4.8	0.023	0.018	0.69
3.35	11.00	0.091	0.245	0.091	1.07	0.097	3.8	0.019	0.020	0.80
5.33	17.50	0.000	-4.162	0.000	1.98	0.000	0.0	0.000	0.000	0.00

(1) Flag "nlp" - based on laboratory testing of fine content, plasticity index, liquid limits and potential intact moisture content.

"nlp" - no liquefaction potential

LDI = 0.22 (m) 8.6 (m) S= 0.038 (m) 1.50 (m)



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